PROCEEDINGS

AMERICAN CONCRETE INSTITUTE

Volume 29—1933

from Journal of the American Concrete Institute
Complete in *Proceedings* Pages from issues for September, October, November, and December, 1932, and January, February, March-April, and June, 1933 (8 Journal issues)

With Index

and

ABSTRACTS

of the current literature of cement and concrete
September and October, 1932

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1933

Contents

American Concrete Institute

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*The five latest, living past presidents are members of the Board

Institute Objects, Organization and Committees

The American Concrete Institute was organized in 1905 (incorporated District of Columbia 1906) as the National Association of Cement Users. In 1913 its charter was amended with change of name. Through the mutual efforts of its membership it maintains a continuous, co-ordinated program in studying the properties of concrete and reinforced concrete and their constituent materials; involving considerations of concrete design, manipulation, application, service. Its aims are to increase the knowledge of concrete and to extend the use of that knowledge. The product of its work is in papers, committee reports and in discussions of them—correlating the experience of many agencies, public and private.

The Institute holds an annual convention as a forum for the presentation and discussion of outstanding developments in the theory and practice of its field, publishes the Journal of the American Concrete Institute monthly, except July and August* The Journal is in two sections: *News Letter*, for the news of the society, informal contributions tending to stimulate its technical program and timely reviews of significant contributions to the literature of cement and concrete, throughout the world. Such reviews of the most significant literature replace *Abstracts*, which were suspended after October, 1932; *Proceedings* for technical papers, reports, discussions.

Proceedings have been published each year beginning 1905; in 1913, 1914, 1915 (Vols. 9, 10 and 11) these were issued periodically in Journal form. A return to the annual volume was made in 1916 with Vol. 12.† After completing Vol. 25 in June 1929, Journal publication was again undertaken. No. 1 of Journal, Vol. 1 (new series) containing the first part of *Proceedings*, Vol. 26 was issued in November, 1929.

The work of the organization is governed by a Board of Direction, formerly with 15 members, in the future with 18 members and administered by the Secretary, appointed by the Board. New By-Laws, ratified by the membership, May 23, 1933, are here published.

The Board divides the labors of formulating details of the creative program among three major committees: Advisory,

*This schedule was modified in this Volume which is complete in 444 *Proceedings* pages from the eight Journals published as follows: September, October, November and December, 1932, January, February, March-April and June, 1933. The schedule for Vol. 30 includes five Journals in alternate months, October, 1933, to June, 1934, (October, December, February, April, June).

†See note next page.

originating technical committee assignments; Program, developing the annual convention; Publications, in general charge of the JOURNAL, which is edited by the Secretary. These committees report to the Board for approval of their undertakings.

In all of this the Institute constitutes a clearing house for the information of and for its chosen field. It is dependent on many public and private agencies and the work of individuals whose studies produce the data for which the Institute offers channels of dissemination and a forum for critical consideration and discussion.

Board of Direction and Executive Committee

The Board of Direction meets at the time of the annual convention; again in the spring (usually early May) and in the Fall (usually the first week of October). Between meetings its authority is vested in the executive committee consisting of the President, Secretary and three other Board members, named by the Board:

S. C. HOLLISTER
F. R. MCMILLAN
A. E. LINDAU
ARTHUR R. LORD
HARVEY WHIPPLE

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†Vol. 11, 1915 is incomplete, numbers 9 to 12 of that year never having been issued. Several numbers of the *Journal* for 1913, 1914 and 1915 are out of print as are also a few of the early volumes of *Proceedings*. Of the *Journals*, Nov. and Dec. 1913; Jan., Feb., Mar. 1914; and Jan., Feb., Mar., Apr., May, June, July and August 1915 are available and are for sale at 50 cents each. A limited supply of some of the annual volumes, to and including Vol. 22, so far as available are for sale to non-members at $8.00 each; Vol. 23, 24 and 25 at $10.00 each. The member price for available volumes to and including Vol. 25, 1929, is $5.00 each.

The JOURNAL is sent to all members (paid for by an allotted portion of the annual dues). Bound volumes 26, 27, 28 and 29 consisting of Proceedings, Abstracts, and Indices are available to non-members at $12.50 each, to members at $6.00 each.

The member price for Vol. 30, completed with the June issue 1934 will be $5.00, subject to a 40 per cent discount on **cash orders before November 15, 1933.** Not more than two such orders for one member. The Institute quotes this special price on orders received at the beginning of each JOURNAL year, because it then knows the actual number of orders to be filled and can provide enough extra copies of each month's JOURNAL to make up the required number for binding. **After November 15, the price will be $5.00 net to members,** because such orders must be filled from a limited reserve stock. The non-member price for Vol. 30 will be $10.00.

Advisory Committee

The Advisory Committee has 12 members: a Chairman, a Secretary, the chairman of each of nine departments of the Committee organization; together with the chairmen of the Program and Publications Committees. Its responsibility is in recommendations to the Board of Direction as to technical committee undertakings.

ARTHUR R. LORD
Chairman

HARVEY WHIPPLE
Secretary

F. R. MCMILLAN
Chairman Department 100, Research

R. W. CRUM
Chairman Department 200, Materials

A. W. STEPHENS
Chairman Department 300, Engineering Design

REXFORD NEWCOMB
Chairman Department 400, Architectural Design

MILES N. CLAIR
Chairman Department 500, Specifications

R. B. YOUNG
Chairman Program Committee and Chairman Department 600, Field Construction

W. D. M. ALLAN
Chairman Department 700, Shop Manufacture

RAYMOND E. DAVIS
Chairman Department 800, Use Requirements

F. H. JACKSON
Chairman Department 900, Joint Efforts

J. C. PEARSON
Chairman Publications Committee

Program Committee

The Program Committee's work is to select from among papers and reports submitted to the Institute such contributions as it deems desirable for the annual convention. The acceptance of contributions by the Publications Committee is entirely independent of the convention and in no way commits the Program Committee as to their use on the convention program. (Likewise, acceptance by the Publications Committee of papers for JOURNAL publication is independent of contributions accepted by the Program Committee for the convention.) It is anticipated that the JOURNAL will publish papers, reports and discussion in excess of the requirements of conventions. The Program Committee's selection of a contribution for the program in no way implies that it is of greater value than others not so selected. Some contributions are better read by members from the printed record; often highly technical con-

tributions of great value are poorly adapted to oral presentation and impromptu discussion. The Program Committee:

R. B. YOUNG
Chairman
HARVEY WHIPPLE
Secretary
RAYMOND E. DAVIS
F. E. RICHART

A. E. LINDAU
F. R. MCMILLAN
ARTHUR R. LORD
Chairman Advisory Committee
J. C. PEARSON
Chairman Publications Committee

Publications Committee

The Publications Committee has the responsibility of the technical publications of the Institute and invites original papers furthering the objects of the society and discussions of published papers and reports. It passes upon the papers and the reports of committees submitted for publication. All contributions are received through the Secretary who is the editor of the JOURNAL and Secretary of the committee. The committee will make all reasonable effort to advise authors promptly as to availability of contributions for publication—subject to consideration by appointed critics to whom manuscript copies are submitted for their advice to the Publications Committee. The Publications Committee:

J. C. PEARSON
Chairman
HARVEY WHIPPLE
Secretary
P. H. BATES
INGE LYSE

BENJAMIN WILK
ARTHUR R. LORD
Chairman Advisory Committee
R. B. YOUNG
Chairman Program Committee

Abstracts

The JOURNAL section of Abstracts (digests of periodical literature of cement and concrete) was a part of the responsibility of the Publications Committee and was suspended after the JOURNAL issue for October 1932. In future, instead of recording very briefly and often inadequately a very considerable number of all contributions to the literature throughout the world, the same range of view will be maintained but with the object of recording only the outstanding and more truly significant additions to our data and our literature.

Technical Committees

The technical committee organization as of July 1, 1933, is as follows:

DEPARTMENT 100—RESEARCH

To comprise all committee work in which the principal interest is in the interpretation of test data or the initiation or direction of tests.

Chairman—F. R. McMillan
Co-Chairmen—H. J. Gilkey and F. E. Richart

Committee 102—Volume Changes in Concrete
Author-Chairman—G. E. Troxell
Raymond E. Davis, H. J. Gilkey,

Committee 103—Effect of Curing Conditions
Author-Chairman—A. A. Anderson

Committee 104—Permeability of Concrete

Author-Chairman—L. G. Lenhardt

Ira L. Collier
Charles H. Scholer
R. H. Schroeder
M. O. Withey

Committee 105—Reinforced Concrete Column Investigation

Chiarman—F. E. Richart

P. H. Bates
R. L. Bertin
Inge Lyse
F. R. McMillan
W. S. Thomson
W. F. Zabriskie

Committee 106—Effect of Vibration on Properties of Concrete

Author-Chairman—R. F. Leftwich

Committee 107—Properties of Job-Cured Concrete at Early Ages

Author-Chairman—Harlan H. Edwards

R. W. Crum
H. E. Davis
W. M. Dunagan
C. A. Hughes
M. B. Lagaard
R. F. Leftwich
T. E. Stanton
M. A. Timlin

Committee 108—Properties of Mass Concrete

Chairman—Raymond E. Davis

P. H. Bates
R. W. Carlson
J. P. Crowdon
E. C. Eaton
F. W. Hanna
W. C. Hanna
G. W. Hawley
D. C. Henny
Louis C. Hill
W. L. Huber
G. W. Hutchinson
S. E. Hutton
H. L. Jacques
F. R. McMillan
S. B. Morris
W. A. Perkins
J. L. Savage
B. W. Steele
G. E. Troxell
A. J. Wiley
C. P. Williams
Hubert Woods

Committee 109—Plastic Flow

Author-Chairman—Raymond E. Davis

Hardy Cross
J. R. Shank

Committee 111—Effect on Concrete of Repeated and Reversed Loads

Author-Chairman—H. H. Scofield

Committee 112—Resistance of Concrete to Impact

Author-Chairman—W. M. Dunagan

Committee 113—Bond Stress

Chairman—F. E. Richart
Secretary—J. R. Shank

H. J. Gilkey
H. F. Gonnerman
Inge Lyse
T. G. MacCarthy
G. A. Maney
Warren Raeder
Albert Smith

Committee 114—Relations of Cement Content, Strength, Water-Ratio and Durability

Author-Chairman—L. W. Walter

F. R. McMillan
R. B. Young

DEPARTMENT 200—MATERIALS

To comprise all committee work in which the principal interest is that of the production and distribution of materials used in concrete work.

Chairman—R. W. Crum
Co-Chairmen—E. O. Sweetser and R. W. Johnson

Committee 201—Aggregate Specifications

Chairman—H. F. Clemmer

R. L. Bertin
A. T. Goldbeck
Fred Hubbard
F. H. Jackson
Herbert F. Kriege
H. J. Love
H. S. Mattimore
N. C. Rockwood
Stanton Walker
L. E. Williams

Committee 206—Welded Wire Fabric as a Reinforcing Material

Author-Chairman—R. D. Bradbury

DEPARTMENT 300—ENGINEERING DESIGN

To comprise all committee work in which the principal interest is that of the structural design or of the supervision of construction of such work.

Chairman—A. W. Stephens
Co-Chairmen—Hale Sutherland and H. N. Howe

Committee 302—Design of Two-Way Slab on Beams

Author-Chairman—E. H. Uhler

F. E. Brown
S. G. Cutler
A. E. Lindau
Arthur R. Lord
Inge Lyse
W. F. Zabriskie

Committee 304—Fire Resistance Standards in Design

Author-Chairman—Nolan D. Mitchell

R. L. Bertin
R. W. Johnson
F. R. McMillan
T. D. Mylrea
R. R. Zipprodt

Committee 305—Structural Loads and Reductions in Design

Author-Chairman—R. L. Bertin

Peter L. Hein
H. N. Howe
George Thompson

Committee 309—Minimum Permissible Thickness of Slabs and Walls

Author-Chairman—D. E. Parsons

Committee 310—Effect of Settlement of Foundation in Reinforced Concrete Structures

Author-Chairman—Hardy Cross

Committee 311—Moment of Inertia of Reinforced Concrete Members

Author-Chairman—F. E. Richart

M. B. Lagaard
G. A. Maney
Hale Sutherland
L. C. Urquhart

Committee 312—Plain and Reinforced Concrete Arches

Author-Chairman—Charles S. Whitney

Clyde T. Morris
M. O. Withey

DEPARTMENT 400—ARCHITECTURAL DESIGN

To comprise all committee work in which the principal interest is that of the architectural design or of sculpture or of the supervision of such work.

Chairman—Rexford Newcomb
Co-Chairman—O. F. Johnson

Committee 404—Composite Concrete Structures

Author-Chairman—J. Morrow Oxley
C. R. Young

Committee 406—Use of Light Weight Concrete in Buildings

Author-Chairman—Frank A. Randall

Frank Brown
W. S. Wolfe

Committee 407—Painting on Concrete Surfaces

Author-Chairman—F. O. Anderegg

P. H. Bates
C. H. Chisholm
A. E. Horn
R. A. Plumb
E. W. Scripture
Howard I. Servey

Committee 408—Recommended Practice for Use of Color in Concrete

Author-Chairman—W. D. M. Allan

J. G. Ahlers
Joseph W. Ayers
C. A. Cooke
Rexford Newcomb
M. R. Rust
Edward M. Waldo

Committee 409—Recommended Practice in Architectural Monolithic Concrete Construction

Chairman—Arthur R. Lord

Secretary—W. E. Hart

John G. Ahlers
William G. Corlett
G. S. Cumming
John J. Earley
Harris Epstein
H. F. Gonnerman
H. M. Hadley
Otis F. Johnson
Rexford Newcomb
E. H. Praeger
Hugh H. Winter

DEPARTMENT 500—SPECIFICATIONS

To comprise all committee work in which the principal interest is that of the production of specifications for material, services or workmanship on behalf of the owner.

Chairman—Miles N. Clair

Co-Chairman—P. J. Freeman

Committee 501—Standard Building Code

Chairman—A. W. Stephens

Secretary—R. R. Zipprodt

R. L. Bertin
H. P. Bigler
R. B. Bradbury
Frank E. Brown
Miles N. Clair
Hardy Cross
H. N. Howe
R. W. Johnson
A. E. Lindau
Nolan D. Mitchell
T. D. Mylrea
F. E. Richart
Hale Sutherland
L. C. Urquhart

Committee 502—Mixing and Placing Concrete in Buildings

Author-Chairman—Arthur R. Lord

A. S. Douglass
A. B. MacMillan
J. Thumley
W. F. Way

Committee 504—Specifications for Ready-Mixed Concrete

Author-Chairman—Miles N. Clair

Stanton Walker
R. B. Young

Committee 505—Reinforced Concrete Chimneys

Author-Chairman—E. A. Dockstader

Kenneth B. Foster
William Jassoy
E. R. Maurer

Committee 506—Concrete Specifications for the Small Job

Author-Chairman—Arthur R. Lord

Theodore Crane
W. F. Way
P. J. Freeman

Committee 508—Concrete Foundations for Steel Buildings

Chairman—J. E. Hough

DEPARTMENT 600—FIELD CONSTRUCTION

To comprise all committee work in which the principal interest is that of monolithic construction, including job equipment, organization, etc.

Chairman—R. B. Young

Co-Chairmen—R. C. Johnson, Nelson L. Doe

Committee 601—Measuring Materials for Concrete

Author-Chairman—R. B. Young

A. A. Levison
H. C. Ross

Committee 603—Design and Operation of Central Mixing Plants

Author-Chairman—Frank I. Ginsberg

Joseph E. Burke
Miles N. Clair
N. D. Crowley
G. L. Lucas
A. W. Munsell
H. F. Thomson
Fred C. Wilcox

Committee 605—Settlement of Concrete In Forms

Author-Chairman—James Lindon

DEPARTMENT 700—SHOP MANUFACTURE

To comprise all committee work in which the principal interest is that of the manufacture and distribution of factory made concrete products.

Chairman—W. D. M. Allan

Co-Chairmen—C. F. Buente, C. A. Wiepking

Committee 701—Standards for Concrete Masonry Units

Chairman—Benjamin Wilk

Secretary—R. E. Copeland

Nolan Browne
Einar Christensen
E. W. Dienhart
C. J. Herzog
D. E. Parsons
George O. Payne

Committee 702—Prevention of Crazing

Author-Chairman—P. H. Bates

F. O. Anderegg
E. D. Boyer
John J. Earley
J. C. Pearson

Committee 704—Cast Stone Standards

Chairman—Louis A. Falco

Secretary—C. G. Walker

P. H. Bates
George J. Eyrick
Herman Fraunfelder
J. W. Lowell
Ernest Rackle
C. Van de Bogart

Committee 708—Recommended Practice for the Manufacture of Concrete Masonry Units

Chairman—P. M. Woodworth

E. W. Dienhart
C. J. Herzog
M. I. McCarty
Eugene F. Olsen
W. P. Stanton
Stanton Walker

Committee 709—Specifications for Concrete Burial Vaults

Chairman—W. D. M. Allan

C. A. Berg
H. F. Hanneman
Carl Head
H. A. Ledyard
F. J. Mead
Harry Samson
J. A. Stuart
F. H. Van Ness
C. A. Wiepking

DEPARTMENT 800—USE REQUIREMENTS

To comprise all committee work in which the principal interest arises from the requirement of unusual or special qualities in concrete on account of its use in less common construction or where subject to extraordinary exposures.

Chairman—Raymond E. Davis

Co-Chairman—W. F. Way

Committee 801—Durability of Concrete

Chairman—A. E. Lindau
Secretary—F. R. McMillan

P. J. Freeman
G. F. Loughlin
E. Viens
L. W. Walter
R. B. Young

Committee 802—Portland Cement Floor Finish

Chairman—John G. Ahlers

W. E. Hart
Millard F. Bird

DEPARTMENT 900—JOINT EFFORTS

To comprise all committee work done by the Institute jointly with representatives from other societies and work in which the principal interest is clearly in two or more of the foregoing departments.

Chairman—F. H. Jackson
Co-Chairman—Cloyd M. Chapman

Committee 902—Concrete Pavement Standards

Chairman—F. C. Lang

E. M. Flemming
L. W. Teller
Herbert F. Faulkner
A. T. Goldbeck
John Wilson
R. L. Fox

Committee 903—Joint Committee on Specifications for Concrete and Reinforced Concrete

Chairman Institute Representation—S. C. Hollister

J. G. Ahlers
F. H. Jackson
Ben Moreell
N. M. Stineman

Committee 904—A. C. I. Representation Sectional Committee on Plastering

J. C. Pearson

Committee 905—A. C. I. Representation Sectional Committee on Specifications for Portland Cement

J. G. Ahlers
E. D. Boyer
W. K. Hatt

Committee 906—A. C. I. Representation Sectional Committee on Methods of Tests of Road Materials

R. W. Crum

Committee 907—A. C. I. Representation Highway Research Board

D. A. Abrams

Committee 908—A. C. I. Representation, Committee on Masonry and Reinforced Concrete of the American Society of Civil Engineers

A. E. Lindau

Committee 909—A. C. I. Representation on Advisory Committee of Merchants Association of New York for New York Building Code

John G. Ahlers

Committee 910—A. C. I. Representation, Highway Research Board Committee on Correlation of Research in Mineral Aggregates

R. L. Bertin
S. C. Hollister

Committee 911—A. C. I. Contact, Building Code Committee of U. S. Dept. of Commerce

Arthur R. Lord

Committee 912—A. C. I. Representation, Sectional Committee on Sieves for Testing Purposes

J. C. Pearson

(Announcements of committee changes, of the completion of committees not now fully organized and of new committees will be made from time to time in the JOURNAL —*News Letter* section.)

By-Laws

American Concrete Institute

ARTICLE I

Members

Section 1. This Institute shall consist of Active Members, Contributing Members, Student Members and Honorary Members, interested in furthering the Institute's objects as set forth in its Charter.

Sec. 2. An Active Member, or a Contributing Member, shall be a person, firm, corporation or society proposed by two members of the Institute and elected by a two-thirds vote of the Board of Direction and shall enjoy all rights and privileges of membership.

Sec. 3. A Student Member shall be a student in residence at a recognized technical or engineering school, proposed by two members and elected by a two-thirds vote of the Board of Direction. A Student Member shall have the rights and privileges of an Active Member, except that he shall not vote nor be eligible for office. His status shall change automatically to that of Active Member on the first anniversary of his membership next succeeding the date on which he ceases to be a student in residence.

Sec. 4. An Honorary Member shall be a person of eminence in the field of the Institute's interest, and/or one who has performed extraordinarily meritorious service to the Institute, nominated by ten members of the Institute and elected by unanimous vote of the Board of Direction, in secret ballot.

Sec. 5. A firm, corporation or society holding membership may name one individual as its representative who shall enjoy all membership rights and privileges.

Sec. 6. Applications for membership on forms prescribed by the Board of Direction, and resignations from membership, shall be presented in writing to the Secretary-Treasurer. Resignations may be accepted only from members whose dues are not more than sixty days in arrears, except by special action of the Board of Direction.

ARTICLE II

Officers

Section 1. The officers shall be a President, two Vice-Presidents, six Regional Directors (one from each of six geographical districts), three Directors-at-large, and the Secretary-Treasurer, who with the five latest, living Past Presidents who continue to be members, shall constitute the Board of Direction.

Sec. 2. The Board of Direction shall from time to time divide the territory occupied by the membership in the six geographical districts to be designated by numbers, each to be represented by a Regional Director, as provided in Section 1.

Sec. 3. The President, Vice-Presidents, Directors-at-large and Regional Directors, and five members of a Committee on Nominations shall be elected by letter ballot of the Institute membership. The Secretary-Treasurer shall be appointed annually by the Board of Direction.

Sec. 4. Before October 1 of each year the Committee on Nominations shall, by letter ballot of its members, nominate candidates for offices to become

vacant at the next annual convention and twenty candidates for membership on the Committee on Nominations which is to serve in the following year and shall transmit the names of all candidates thus nominated to the Secretary-Treasurer of the Institute. The consent of each candidate for office must be obtained before notice of his nomination is published. The Secretary-Treasurer shall cause notice of all such nominations to be transmitted to the membership of the Institute at least 120 days prior to the next ensuing annual convention. Upon petition to the Board of Direction signed by at least ten members of the Institute, additional nominations for offices or for membership on the Committee on Nominations may be made within 30 days thereafter.

The complete list of nominations shall be submitted 60 days before the next convention to the Institute membership for letter ballot to be canvassed at 5 p. m. on the first day of the convention and the result announced at a session of the convention on the second day. The candidate for any office receiving the most votes shall be declared elected and the candidate receiving the most votes for membership on the Committee on Nominations shall be Chairman of that committee; the four next highest shall be declared elected members of the Committee. With these five the two latest past president members of the Board of Direction shall serve, making a total membership of seven.

Should any member of the Committee on Nominations thus chosen fail, within fifteen days of formal notice from the Secretary-Treasurer, to make written acceptance of service, a vacancy shall occur to be filled by the candidate receiving the next greatest number of votes and so on until the seven places on the committee shall be filled.

Within ten days after the ratification of these By-Laws, the Secretary-Treasurer shall so notify the members of the Nominating committee last elected and the two latest Past-President members of the Board of Direction, and these shall constitute a Nominating committee for the current calendar year with duties prior to the next succeeding election as hereinbefore provided.

Sec. 5. The terms of office of the President, Vice-Presidents, Secretary-Treasurer and Regional Directors shall be one year. This provision shall in no way interrupt terms of officers elected prior to the ratification of these By-Laws by letter ballot of the membership. Directors-at-large shall be elected for terms of three years except that in the election following the ratification of these By-Laws one Director-at-large shall be elected for one year, one for two years, and one for three years, and thereafter annually one Director-at-large for a term of three years. A year is to be here construed as the period between annual conventions.

Sec. 6. The President, Vice-Presidents and Regional Directors shall be ineligible for more than one re-election to the same office until the lapse of at least one term.

Sec. 7. The term of each officer shall begin at the close of the annual convention at which he is elected and shall continue until a successor is duly elected.

Sec. 8. A vacancy in the office of President shall be filled by the Vice-President having Institute membership seniority.

Sec. 9. Vacancy in any office, for the unexpired term, shall be filled by appointment by the Board of Direction except as provided in Section 8.

Sec. 10. In the event of disability or neglect in the performance of his duty of any officer of the Institute, the Board of Direction shall have the power to declare the office vacant.

Sec. 11. The Board of Direction shall have general supervision of the affairs of the Institute and at a meeting held in the period of the annual convention shall appoint a Secretary-Treasurer for a term of one year. It shall name three of its own members as a Finance Committee to serve one year beginning July 1 or until their successors are appointed. The Board shall create Advisory,

Program and Publications Committees, and shall authorize such special committees as it may deem desirable for the administrative and technical work of the Institute, appointing a Chairman for each such committee. Additional committee members shall be appointed by the President.

Sec. 12. There shall be an Executive Committee of the Board of Direction consisting of the President, Secretary-Treasurer, and three of its members appointed by the Board of Direction.

Sec. 13. The Executive Committee shall manage the affairs of the Institute during the interim between the meetings of the Board of Direction.

Sec. 14. It shall be the duty of the Finance Committee to review and amend an annual budget tentatively prepared by the Secretary-Treasurer for each fiscal year commencing July 1, and to recommend, as to proposed expenditures, to the Board of Direction. The accounts of the Secretary-Treasurer shall be audited annually.

Sec. 15. It shall be the duty of the Advisory Committee to make recommendations to the Board of Direction as to technical committee undertakings and the appointment of chairmen, and to recommend to the President the appointment of technical committee members. The Secretary-Treasurer of the Institute shall be Secretary of the Committee and with the Chairman constitute its executive group in intervals between meetings.

Sec. 16. It shall be the duty of the Program Committee to recommend for approval of the Board the annual convention program and to undertake the responsibility of carrying the program into effect. The Secretary-Treasurer of the Institute shall be Secretary of the Committee, and with the Chairman constitute its executive group in intervals between meetings.

Sec. 17. It shall be the duty of the Publications Committee to have supervisory charge of Institute technical publications and to report to the Board of Direction. The Secretary-Treasurer of the Institute shall be Secretary of the Committee, and with the Chairman constitute its executive group between meetings.

Sec. 18. The President shall perform the usual duties of the office. He shall preside at the annual convention, at the meetings of the Board of Direction and of the Executive Committee, and shall be ex-officio member of all committees. He may name a chairman to serve in his place for any sessions of the convention.

The Vice-Presidents, each in the order of his Institute membership seniority, shall discharge the duties of the President in his absence. In the absence of President and both Vice-Presidents a President Pro-Tem, appointed by the Board, shall discharge such duties.

Sec. 19. The Secretary-Treasurer shall perform such duties, furnish such bond and receive such salary as shall be determined by the Board of Direction.

ARTICLE III

Meetings

Section 1. The Institute shall meet annually. The time and place shall be fixed by the Board of Direction. Notice of this action shall be mailed to all members at least thirty days previous to the date of the convention.

Sec. 2. The Board of Direction shall meet at least twice each year, the time and place fixed by the Board.

Sec. 3. The Executive Committee shall meet on call of the President or of any three of its members.

Sec. 4. Twenty-five members shall constitute a quorum for meetings of the Institute; eight members shall constitute a quorum for meetings of the Board of Direction; and three members for meetings of the Executive Committee.

ARTICLE 4

Dues

Section 1. Dues for the several membership classes shall be payable annually in advance of the first of the month following notification of the member applicant of his election by the Board of Direction as follows: Contributing members, $50.00; Active members, individuals, $10.00; firms, corporations or societies, $20.00; Student members, $4.00; Honorary members, none. Any individual member may be admitted to life membership upon payment of a sum determined by the Executive committee based on 90 per cent of the membership dues as established at the time of application, credited with 3 per cent interest compounded annually for the applicant's life expectancy as arrived at from the American Experience Table of Mortality. The changes in dues rates herein provided shall be effective from the first of the month following the ratification of these By-Laws.

Sec. 2. A member shall be entitled to receive one copy of each issue of the JOURNAL of the AMERICAN CONCRETE INSTITUTE as issued in the period of his membership and additional or other publications as determined by the Board of Direction.

Sec. 3. A member whose dues remain unpaid for a period of six months shall forfeit the privileges of membership and shall be officially notified to this effect by the Secretary. If these dues are not paid within six months thereafter his name shall be stricken from the list of members, unless otherwise specifically ordered by the Board of Direction. Members may be reinstated upon payment of all indebtedness against them upon the books of the Institute.

ARTICLE V

Standards

Section 1. Proposed new or revised standard specifications, standard practice and standard definitions when approved by a majority voting in the committee in which they originate shall be submitted in a form approved by the Publications committee to the Secretary-Treasurer of the Institute at least one hundred twenty days prior to the opening of the annual convention at which the committee proposes to present them for approval. They shall be accompanied by a supplementary report by the committee constituting a summary or outline of the scope of the proposed standards of such length and detail as the Publications committee may require. The Secretary-Treasurer of the Institute shall cause the outline or summary to be printed and mailed to the whole membership of the Institute at least thirty days prior to the opening of the convention. Subsequently, the Secretary-Treasurer shall mail to any member on his specific request a copy of the complete recommendations of the committee. At the subsequent convention they may be amended and approved by a majority vote giving them tentative adoption. Notice of such convention action shall be published in the next succeeding issue of the JOURNAL within sixty days thereafter and the Secretary-Treasurer shall mail to any member on his specific request a complete record of the report as revised and tentatively adopted. At a subsequent annual convention the tentative standard may again be offered unamended and as there approved by a majority of those voting shall be submitted to the Institute membership for approval. Notice of such action shall be published in the JOURNAL within sixty days thereafter. Such proposed standards shall be considered adopted unless within one hundred twenty days subsequent to such notice protests are received from as many Institute members as there were members of the originating committee.

Proposed standards may be given the status of tentative adoption in an interval between conventions by approval of two thirds of the combined membership of Advisory and Publications committees upon evidence of general consensus of opinion in the committee originating the report and after due consideration of any protests within one hundred twenty days subsequent to publication of notice of its availability for distribution.

ARTICLE VI

Amendments

Section 1. Proposed amendments to these By-Laws, signed by at least fifteen members, if presented in writing to the Board of Direction ninety days before the annual convention, shall be mailed to the membership at least thirty days prior to the annual convention. These amendments may be discussed and amended at the annual convention and be passed to letter ballot by a two-thirds vote of those present. Two-thirds of the votes cast by letter-ballot canvassed within ninety days thereafter shall be necessary for their adoption.

A. C. I. Standards

Titles of current Standards, and of Tentative Specifications and Proposed Specifications and Recommended Practice of the Institute and their serial designations are here listed with reference to their publication in the Proceedings of the American Concrete Institute.

Curb and Gutter

Tentative Specifications for Concrete Curb and Concrete Curb and Gutter, S 6-E-27T. Vol. 23, p. 684.

Masonry Units

Standard Specifications for Concrete Building Block and Concrete Building Tile, P 1-A-29. Vol. 23, p. 696—revised—see tentative amendment, Vol. 24, p. 834. Adopted as standard by letter ballot. Note: Vol. 25, p. 605. This standard as amended and adopted appears as an Appendix to report of Committee 708, Vol. 27, p. 1017 (April 1931 Journal).

Standard Specifications for Concrete Sewer Manhole and Catch Basin Block, P 1-C-29. Vol. 23, p. 694. Adopted as standard by letter ballot. Note: Vol. 25, p. 605.

Standard Specifications and Building Regulations for Concrete Staves, P 4-A-26. Vol. 22, p. 666.

Tentative Specifications for Concrete Brick, P 1-B-26T. Vol. 21, p. 604—revised—see tentative amendment, Vol. 22, p. 667—and Note: Vol. 23, p. 690.

Tentative Recommended Practice for the Manufacture of Concrete Building Block, Building Tile and Brick, P 6-A-25T. Vol. 21, p. 473.

Proposed Recommended Practice for the Manufacture of Concrete Building Block and Tile (progress report of Committee 708 presented for discussion with a view to superseding Tentative Recommended Practice for the Manufacture of Concrete Building Block, Concrete Building Tile and Brick, P 6-A-25T—see above). Vol. 27, p. 1001 (April 1931 Journal). Discussion Vol. 28, p. 155 (Oct. 1931 Journal).

Tentative Specifications for Cast Stone, P 3-A-29T. Vol. 25, 1929. See amendment recommended by Committee 704, Vol. 28, p. 33 (Sept. 1931 Journal).

Proposed Recommended Practices in the Use of Cast Stone (Report of Committee 704, presented for discussion). Vol. 26, p. 760 (May 1930 Journal).

Pipe and Drain Tile

Standard Specifications for Concrete Drain Tile, P 7-B-25. (As tentatively published Vol. 20, p. 678.)

Tentative Specifications for Reinforced Concrete Sewer Pipe, P 7-C-25T. Vol. 21, p. 584.

Tentative Specifications for Plain Concrete Sewer Pipe, P 7-A-24T. Vol. 20 p. 669.

Tentative Specifications for Reinforced Concrete Culvert Pipe. Vol. 25, 1929, p. 608. (Second report. Adopted by the Joint Concrete Culvert Pipe Committee representing American Concrete Institute (Com. 901); American Association of State Highway Officials; American Concrete Pipe Association; American Railway Engineering Association; American Society for Testing Materials; American Society of Civil Engineers; U. S. Department of Agriculture.)

Reinforced Concrete

Tentative Building Regulations for Reinforced Concrete, E 1-A-28T. Vol. 24, p. 791.

Standard Specifications for Concrete and Reinforced Concrete. Vol. 21, p. 329. (Adopted by the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete representing American Concrete Institute (Com. 903); American Society of Civil Engineers; American Society for Testing Materials; American Railway Engineering Association; Portland Cement Association.)

Tentative Construction Specification for Concrete work on Ordinary Buildings, 502-31T. Vol. 26, p. 1 (November 1929 JOURNAL); discussion Vol. 26, p. 580 (March 1930 JOURNAL); Vol. 27, p. 99 (September 1930 JOURNAL); amended and adopted as tentative specification Vol. 27, p. 1181 (May 1931 JOURNAL).

Tentative Construction Specification for Concrete Work on the Small Job, 506-31T. Vol. 27, p. 65 (September 1930 JOURNAL); discussion, Vol. 27, p. 525 (January 1931 JOURNAL); amended and adopted as tentative specification, Vol. 27, p. 1184 (May 1931 JOURNAL).

Specification for Supplying, Fabricating and Setting Reinforcing Steel on Ordinary Buildings, 503-32, with an appendix: "A Steel Setters' Primer." Vol. 26, p. 444 (February 1930 JOURNAL); discussion Vol. 26, p. 910 (June 1930 JOURNAL); adopted as tentative specification Vol. 27, p. 1186 (May 1931 JOURNAL). (Referred to letter ballot for adoption as standard April 1932—adopted.)

Roads

Standard Specifications for One-Course Concrete Pavement for Highways, S 6-A-28. (Vol. 20, p. 695—adopted as standard, with revisions, Vol. 24, 1928, p. 852.)

Standard Specifications for Two-Course Portland Cement Concrete Pavement for Highways, S 6-B-28. (As tentatively published Vol. 20, p. 710—adopted as standard, with revisions, Vol. 24, 1928, p. 852.)

Standard Specifications for One-Course Portland Cement Concrete Street Pavement, S 6-C-28. (As tentatively published Vol. 20, p. 716—adopted as standard, with revisions, Vol. 24, 1928, p. 852.)

Standard Specifications for Two-Course Portland Cement Concrete Street Pavement, S 6-D-28. (As tentatively published Vol. 20, p. 724—adopted as standard, with revisions, Vol. 24, 1928, p. 852.)

Proposed Specifications for Concrete Pavement in Municipalities, with an appendix. Report of Committee 902, Vol. 28, p. 453 (March 1932 JOURNAL).

Sidewalks and Floors

Standard Specifications for Portland Cement Concrete Sidewalks, C 2-B-25. Vol. 21, p. 591.

Standard Specifications for Concrete Floors, C 2-A-24. Vol. 20, p. 739.

Proposed Recommended Practice for Heavy Duty (concrete) Floor Finish with Notes on Light Duty Floor Finish
and

Proposed Recommended Practice for Dusted-on (concrete) Floor Finish (embodied in report of Committee 802 presented for discussion) Vol. 26, p. 520 (March 1930 JOURNAL) and discussion Vol. 27, p. 115 (September 1930 JOURNAL).

Surfaces

Standard Recommended Practice for Portland Cement Stucco. C 3-A-23. Vol. 19, p. 471.

Tentative Recommended Practice for Treatment of Exterior Surfaces of Industrial Reinforced Concrete Buildings, C 3-B-25T. Vol. 21, p. 564.

Tentative Specifications, Finish Coat, Portland Cement Stucco, C 3-C-29T. Vol. 25, 1929.

Proposed Recommended Practice for the Use of Pigment Admixtures in Troweled Concrete Surfaces—progress report of Committee 408, presented for discussion, Vol. 27, p. 975 (April 1931 JOURNAL). Discussion Vol. 28, p. 151 (Oct. 1932 JOURNAL).

Miscellaneous

Standard Methods for the Measurement of Concrete Work, C 5-A-26. Vol. 22, p. 655.

Tentative Purchase Specifications for Concrete Aggregates, E 5-A-29T. Vol. 25, 1929.

Standard Specifications for Monolithic Concrete Sewers and Recommended Rules for Sewer Design, S 3-A-24. Vol. 20, p. 757.

Standard Definitions, G 4-A-23. Vol. 19, p. 319.

Tentative Specification for Ready-Mixed Concrete, 504-31T. Vol. 27, p. 1177 (May 1931 JOURNAL).

Tentative Specification for Concrete Burial Vaults, 709-32T, Vol. 28, p. 633 (May 1932, JOURNAL).

The Institute has done other work in standardization involving specifications and recommended practice now regarded as obsolete or not brought to date, or progress reports which are incomplete and for these reasons not here listed.

Medals and Awards

The Wason Medals

Founded by Leonard C. Wason, Boston, past-president, American Concrete Institute.

Awards, 1917-1933

For the most meritorious paper presented at each annual convention.

1916 Paper—A. B. McDaniel, "Influence of Temperature on the Strength of Concrete."

1917 Paper—Charles R. Gow, "History and Present Status of the Concrete Pile Industry."

1918 Paper—Duff A. Abrams, "Effect of Time of Mixing on the Strength and Wear of Concrete."

1919 Paper—W. A. Slater, "Structural Laboratory Investigations in Reinforced Concrete Made by Concrete Ship Section, Emergency Fleet Corporation."

1920 Paper—W. A. Hull, "Fire Tests of Concrete Columns."

1921 Paper—H. M. Westergaard, "Moments and Stresses in Slabs."

1922 Paper—George E. Beggs, "An Accurate Mechanical Solution of Statically Indeterminate Structures by Use of Paper Models and Special Gauges."

1923 Paper—John J. Earley, "Building the Fountain of Time."

1924 Two Papers—Richard L. Humphrey, "Twenty Years of Concrete" and "The Promise of Future Development."

1925 Paper—E. A. Dockstader, "Report of Tests Made to Determine Temperatures in Reinforced-Concrete Chimney Shells."

1926 Paper—A. Burton Cohen, "Correlated Considerations in the Design and Construction of Concrete Bridges."

1927 Paper—Arthur R. Lord, "Notes on Concrete—Wacker Drive, Chicago."

1928 Paper—Franklin R. McMillan, "Concrete Primer."

1929 Paper—L. G. Lenhardt, "The Concrete Lining of Detroit Water Tunnels."

1930 Paper—I. E. Burks, "Concreting Methods at the Chute a Caron Dam."

1931 Paper—Raymond E. Davis and Harmer E. Davis, "Flow of Concrete under the Action of Sustained Loads."

1932 Paper—Charles S. Whitney, "Plain and Reinforced Concrete Arches."

Awards for Research, 1929, 1930, 1932 and 1933

S. C. Hollister, for advancing the art of bridge construction in the design, construction and test of a concrete skew arch as reported in his 1928 paper: "The Design and Construction of a Skew Arch."

H. F. Gonnerman and P. M. Woodworth for the work reported in their 1929 paper: "Tests of Retempered Concrete."

M. O. Withey, for the work reported in his 1931 Institute paper, "Long Time Tests of Concrete."

T. C. Powers, for work reported in his 1932 Institute paper, "Studies of Workability of Concrete."

The Turner Medal

Founded 1927 by Henry C. Turner, New York, past-president, American Concrete Institute, a gold medal will be awarded not oftener than once a year "for notable achievement in or service to the concrete industry."

AWARDS, 1928, '29, '30 AND '32

Awarded, 1928, to ARTHUR N. TALBOT, for "outstanding contributions to the knowledge of reinforced concrete design and construction."

Awarded, 1929, to WILLIAM K. HATT, for "pioneer work in reinforced concrete research; for a quarter century of devoted, outstanding and continuous service in developing the knowledge of concrete."

Awarded, 1930, to F. E. TURNEAURE, for "distinguished service in formulating sound principles of reinforced concrete design."

Awarded 1932 to DUFF A. ABRAMS, for "The discovery and statement of important fundamental principles governing the properties of concrete and reinforced concrete."

INDEX

PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

VOLUME 29—1933

(from JOURNAL, Vol. 4, Sept. 1932 to June 1933)

In general, important subjects are classified and indexed under approximately 30 main headings each one appearing in its proper alphabetical order in bold face capital letters—as for instance, ARCHITECTURAL DESIGN. "Surface treatment" and other subjects classified under this head, are indented.

Key words to important subjects appear, in alphabetical order in addition to the general classification—as for instance "Admixtures" and "Blast furnace slag," each referring to MATERIALS AND TESTS under which all allied references appear, indented. Authors' names appear in proper alphabetical sequence with the subjects, with references to their contributions.

Specific data on Beams are so indexed by reference to "ENGINEERING DESIGN" or "TESTS OF MEMBERS AND COMPLETED STRUCTURES" thus avoiding an oversight by the searcher of important allied data.

The readiest use may be made of this index by gaining some familiarity with the thirty main classifications.

The searcher for information on design and proportioning of concrete mixtures will look for MIXTURES and refer to subheads; "Design" and "Proportions." Field control methods will be found under TESTS AND FIELD CONTROL.

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DISCUSSION PLAIN AND REINFORCED CONCRETE ARCHES

In equation No. 8, 12th line from top of page 90, Capital "C" in denominator of second term should be lower case "c". In 17th line from top of same page, greek letter $\propto$ should be ∞ symbol for infinity.

Painting on Concrete Surfaces

*Report of Committee 407**

BY F. O. ANDEREGG, AUTHOR-CHAIRMAN†

This report has been compiled from information received from a large number of manufacturers, users and others interested. It aims to give fair and concise information concerning: (1) the effect of factors arising from the concrete and the nature of its surface, (2) the various painting materials that are being used and (3) the best probable practice in their application to produce satisfactory decorative coatings. The subject of waterproofing as such is not considered, although any paint applied to exteriors must show reasonable resistance to the elements.

The colors, sometimes in attractive combinations, which may be obtained with most of the surface treatments, afford the architect suitable means of expression.

THE CONCRETE

Surface Conditions

"A paint is no stronger than its foundation." If the surface to be painted is covered with laitance, form-oil, soot, loosely held material or efflorescent salts, it is useless to expect satisfactory results with any paint, unless the surface is first vigorously wire brushed or lightly sand blasted. Mechanical anchorage improves the strength and durability of the film very greatly and where the surface is glassy, from excessive trowelling, it is often helpful to remove part of the glaze.

The question of removal of form marks is determined by the architectural effect desired, while sand streaks, honeycombing or tie-wire holes should first be cut to a depth of ¾-in. with square shoulders and painted with neat cement slurry. They should then be plastered

†Consulting Specialist on Building Materials, Pittsburgh, Pa.

*It is noteworthy that this report was produced promptly by Dr. Anderegg, the Author-Chairman of the Committee; that it is brief, that it has in general the approval of all critic members of the committee, four of them representing paint producers: A. E. Horn, R. A. Plumb, E. W. Scripture and Howard I. Servey and two of them respresenting user interests: P. H. Bates and C. H. Chisholm and by several non-members of the Committee to whom the author submitted it, prior to presenting it for publication. "Wherever differences of opinion have arisen, as is to be expected where information is received from manufacturers of competing materials, I have endeavored to make all statements as fairly as possible," writes Dr. Anderegg. "The unqualified approval generally received seems to indicate a treatment fair to all and without bias."—Editor

with a slight excess of mortar, using as much pressure as possible. After the cement has partly hardened the excess should be removed and the patch finished with a carborundum stone to match the rest of the surface.

Suction

The suction to be left in the concrete is of importance. With hydraulic paints the absorption should be largely satisfied so as to provide ample curing moisture. With oil paints and lacquers, the concrete must be quite dry and, if very porous, the first coat should be suitably diluted with vehicle and solvent; many prefer to use vehicle only.

Green Concrete

Concrete recently placed, is more or less saturated with *moisture* and in a highly *alkaline condition.* These facts impose fundamental limitations which must be recognized in securing successful painting.

Moist concrete

The greater affinity of portland cement and the common concrete aggregates for water than for vegetable oils, makes it poor practice to try to apply paints of largely organic character to moist concrete. Moreover, as a result of this greater affinity, moisture penetrating concrete from behind will often bodily push an oil-paint film from the concrete. This phenomena is called, very expressively, "preferential wetting". If the paint film is permeable, evaporation may take place so that the film is not shoved off. A good test for dryness is to place a sheet of linoleum or glass against the concrete, observing after 24 hours any condensation of moisture.

Alkalies in concrete

Portland cement on hydrating releases considerable lime as hydroxide. Any moisture present is saturated with this compound. As air comes in contact therewith, it is slowly carbonated and rendered relatively inert. In addition, the cement contains sodium and potassium compounds which are set free partly as very *caustic* hydroxides. These alkali metal compounds react with vegetable oils and some guns saponifying them and radically changing their character. Even after carbonation, they are alkaline and still harmful. They are very soluble, however, and as the concrete dries out, are brought to the surface where they are readily removed by the rain or by systematic washing. In other words, they are more or less fugitive.

Various methods have been proposed for taking care of the alkali in the concrete: treatments with zinc sulfate, alum or magnesium fluosilicate. Some people prefer the first, but the last seems generally regarded as best and has the advantage, probably, of partial filling of

the pores in the surface of the concrete. It may be obtained under various brand names. This sort of treatment, while helpful, cannot be considered as completely efficacious.

Efflorescent crystal pressure

It is difficult satisfactorily to paint a concrete within which an appreciable quantity of soluble salts has accumulated.

Where alkali-metal compounds are not almost completely removed; where concrete is in contact with the soil so that the soil solution diffuses into the concrete; where it is in contact with cinders, or with floors in dairies, food factories, pickle factories or packing houses; or where rain water has had prolonged opportunity to build up the sulfate content of the concrete; it is extremely difficult to secure a satisfactory coating of paint. The molecular forces involved in oriented crystal growth are tremendous, involving hundreds of thousands of pounds per square inch. While the force exerted by a single cyrstal is not great because of its small area, the cumulative effect of the millions found in many concretes will push off most paint films.

Proper precautions should be taken in the design by the use of proper flashing, calking and waterproofing to prevent moisture entering the concrete which is to be covered with any treatment other than hydraulic cement paints.

PAINTING MATERIALS

A variety of paints have been applied to concrete ranging from cold water paints to nitrocellulose lacquers. Each material has its limitations, but some are much better than others. They fall into two large groups; those largely inorganic and those largely organic in composition.

The *permeability* of the paint film is very important in determining its successful use under various conditions. There are two schools of thought: One holds that moisture must be kept out of the concrete, insists on having a film as nearly impervious as possible; while the other school, realizing that moisture very often gets into the concrete from behind, believes opportunity should be provided for the wall to "breathe." If moisture has a chance to escape through pores in the surface paint, the film will not be pushed off as a whole by preferential wetting.

The ability of the paint material to transmit the salts in solution in the moisture affects the pushing off of the film by efflorescent crystal pressure. Damage will depend on whether the crystals grow on the outside of the film or back of it.

A. Largely inorganic

These treatments are not generally applied with success to surfaces which have been painted with oil paints unless the latter have been completely removed.

1. *Cold water paints,* which usually contain lime with glue or casein as the cementing agent, together with pigments. Sometimes portland cement is added, but its hardening properties are reduced by the glue or casein. These paints are not suitable usually for exterior use, and where the concrete base gets damp, their durability is low, due to the decay of the casein or glue. Their use should be limited to dry, interior walls. These films are probably less permeable than those of cement paint so that the accumulation of salts near the concrete surface often pushes the paint film off.

2. *Portland cement or hydraulic paints,* which contain a large amount of portland cement, suitable pigments and other ingredients. Some of these are diluted with more or less lime. The effect of the lime is to reduce the durability and to decrease the opacity when wet. On the other hand, the ease of spreading is slightly increased, as is the flexibility of the paint film. Sometimes organic waterproofing materials are added, which greatly increase the ease of spreading. If stearate is used the durability is probably not impaired, but the effect of other waterproofing admixtures in this regard has hardly been satisfactorily demonstrated.

Portland cement paints are suitable for inside and outside application to walls, but are not recommended for painting floors except the floors of swimming or wading pools. These paint films offer reasonable resistance to the weather, often effecting a perceptible improvement in this regard over the original surface. These films are not impermeable to the passage of moisture, so that a wet wall has a chance to "breathe" or dry out. Any efflorescent salts in the concrete tend to pass through the cement film to be deposited on the surface without much damage to the paint.

3. *Proprietary chemical treatments,* whereby colored stains, mostly inorganic, are deposited within the pores and over the surfaces of the concrete. Very beautiful effects are claimed for these methods, but their best application would seem to be for interiors, for most of the compounds deposited are susceptible to weathering, resulting in gradual fading. The permeability of the surface to moisture is reduced by this treatment by an estimated one half to three fourths, but the nature of the deposit does not lead to probable detachment by preferential wetting or crystal pressure. Unless concrete in contact with the

ground or other source of moisture has been waterproofed, a deposit of efflorescent salts often causes a fading of the colors.

B. Largely organic

These paints are best applied in warm, dry weather.

1. *Oil paints*, which are composed of a vegetable drying oil vehicle, pigment and sometimes solvents or gums. The oils vary from linseed, which is probably most easily saponified by the alkalies found in concrete, to tung (china wood) oil, which seems to be least affected by these reagents. Hence the increasing popularity of the latter and the efforts to produce this oil in the southern part of this country. The former seems to be, however, more durable against weathering.

The gums may vary from rosin through the rosin esters, the fossil gums to some of the new condensation resins. Sometimes these require special plasticizers and solvents which are added in conjunction with the drying oil. The gums vary in their degree of resistance to the action of alkalies and to weathering. The permeability of these drying-oil films is slight so that the film may be bodily displaced if the concrete becomes wet. When solvents are used, it is difficult to fill completely the larger pores in the concrete.

Paints containing aluminum powder alone or in conjunction with other pigments have been applied with apparently satisfactory results on a number of jobs. The reflecting power of the aluminum reduces the deterioration of the film by light, but on the other hand, the presence of the leafed-out metal probably lessens the opportunity of moisture escaping from the concrete. A bituminous-aluminum paint has been developed for giving a decorative finish to a bituminous waterproofing treatment of concrete. Another, somewhat similar treatment, involves alternate coats of vegetable drying oil and micaceous sands.

2. *Lacquers*, which usually contain synthetic plastics as bases, suspended in suitable solvents with the aid of plasticizers. Very often all of the materials entering these lacquers are synthetic. The most notable examples of these plastics are cellulose nitrate or acetate. These have apparently low permeability and therein seems to lie their danger, for the whole film is readily pushed off the concrete surface by moisture in the concrete. Glyptol and phenol resins are also frequently utilized in this way.

Certain of the vinyl resins have been used with fairly good success on concrete. Some of the films allow moisture to evaporate to some extent and when properly applied become mechanically well bonded to the surface. They have, however, a tendency to darken a bit on exposure

to light. A number of other materials are coming on the market but opportunity to make adequate tests has been lacking.

3. *Hybrides.* In order to get better bond between the cellulose esters and concrete surfaces sometimes vegetable oils or some of their fatty acids are added. These admixtures greatly increase the rate of penetration into the concrete, allowing the film to secure a much more effective mechanical anchorage with marked improvement in quality, but the corrosive action of unsaturated fatty acids on portland cement products must not be disregarded.

4. *Conclusions.* Where the concrete is quite dry and free from moisture ingress from behind, an oil paint of a high degree of impermeability may be used to advantage: but where moisture is likely to get into the concrete, a paint that is porous enough to permit the moisture to evaporate is apt to give better satisfaction.

METHODS OF APPLICATION

The wide variation in nature of the painting materials applied to concrete requires special methods of application. Several points having to do with application have already been discussed. A few additional methods need discussing.

Generally speaking, the paints of largely organic character may not and must not be applied to damp concrete surfaces. The concrete should have aged until the alkaline reaction has been largely neutralized or some treatment such as fluosilicate should have been given it and it should be allowed to dry out pretty thoroughly. The importance of securing adequate mechanical anchorage should be emphasized; a priming coat should be selected which penetrates well down into the concrete.

Coldwater paints are generally applied to a surface more or less dry; as their setting-up depends mostly upon a drying action, the presence of excess moisture is not desirable.

For the production of chemically precipitated color treatments the concrete should not be moist because the action depends upon drawing of the solutions down into the pores.

With portland cement paints, mechanical anchorage is not so essential because the cement bonds to the surface in the same way in which the concrete itself has hardened, but a highly troweled, smooth surface should be roughened a bit to give proper bond.

To secure the best possible results with portland cement paint the wetting of the concrete surface and the subsequent curing of each coat of paint are highly important and must be carefully carried out. The following points are essential:

The surface must be moistened before the paint is applied and the first coat of paint should be wet down as soon as possible after the paint is hardened sufficiently to prevent injury to the surface.

Before the second coat is applied the first coat should be dampened by spraying. The second coat of paint should be moistened with a fine spray as soon as it is hardened sufficiently to prevent the surface from being marred and should be kept wet as long as practicable.

During warm, hot and windy weather the paint film will harden or "set" quicker than during cool or cold weather, so that the paint film will need to be sprayed during the warmer weather sooner than during the colder weather. In order to get best results under these conditions it is wise to make the application in the evening.

Readers are referred to the JOURNAL *for January 1933, for discussion which may develop. Such discussion should reach the Secretary by November 1, 1932.*

The Mortar Voids Method of Designing Concrete Mixtures*

BY MARK MORRIS†

The selection of a combination of materials to compose the batch which will produce concrete of a given unit strength offers one of the most fascinating problems associated with the use of concrete. To the designer of the concrete mixture, it is something of an adventure, for there is, as yet, no well-defined, thoroughly dependable and generally accepted method of solving the problem. Even though he must attain his objective by following some devious path rather than traveling a broad, safe, well-marked highway leading directly to it, he has available for his use many landmarks and direction signs which have been erected by those who have preceded him. These guide posts establish limits, or boundaries, and mark danger points. A combination of art and science enables him to chart his way through the forest of possibilities to the selection of the one or several proportions which will most probably be satisfactory. Final selection, consisting principally of slight adjustments of the chosen proportion, must usually await results of strength tests upon the concrete as produced under actual construction conditions. The suspense attending the approach of this climax contributes considerably to the fascination of the problem.

Much has been accomplished toward the provision of a method for designing concrete mixtures, and much remains to be done. Considerable success has attended the design of mixtures for the preparation of laboratory specimens and for the fabrication of concrete on the job, but the effect of differences between the field and the laboratory in the mixing, placing and consolidating operations upon the composition of the concrete requires assumptions during the progress of the designing that call for considerable experience on the part of the designer, with the class of concrete to be made. A particularly difficult problem is the predetermination of the quantity of water re-

*Presented at 28th Annual Convention, March 1-4, 1932, Washington.
†Research Assistant, Department of Materials and Tests, Iowa State Highway Commission, Ames, Iowa.

quired in the batch. Unless this can be known with some accuracy, the estimate of the quantities of the solid materials required will vary from the actual quantities by more than the percentages considered as satisfactory tolerances, that is, about two per cent for the cement, four to seven per cent for the sand, and three to five per cent for the coarse aggregate. To establish values which must now be assumed, the research worker should go to the field and become as familiar with the composition of the fresh concrete deposited under actual job conditions as he is with that in the laboratory specimens. To a limited extent, this has been done. When the recorded experiences of a number of investigators for a considerable range of conditions have become available and have been studied, it seems likely that rapid progress will be made toward the goal so long sought, that of designing, after a brief examination of the available materials, a concrete mixture that can be used with very slight adjustment, if any, when taken to the field or job to produce concrete having at a given age, within five per cent of the desired unit strength, and containing well within the limits of the satisfactory tolerances the estimated quantities of materials, including water.

During the last quarter of a century many researches in concrete have had as their objectives the discovery of the basic laws, principles or relationships existing between combinations of the essential materials and the unit strength of concrete. Invaluable contributions to the store of knowledge regarding the composition of concrete have been made. Some of these researches have revealed sufficiently portions of what appear to be the basic laws outlining the relationship between the composition of concrete and its unit strength to permit the use of a method, or of methods, of designing concrete mixtures that may be based on this knowledge. It is the purpose of this paper to describe some of the experiences of the author with the study and use of facts revealed by one of the more comprehensive of these investigations, the work of Talbot and Richart, an account of which was published October, 1923, in the University of Illinois, Engineering Experiment Station, Bulletin Number 137.

The method developed by Talbot and Richart for obtaining data to determine the concrete-making ability of different sands has been used by the Iowa State Highway Commission to acquire detailed knowledge of the characteristics of a large number of sands available for use in Iowa. Circumstances surrounding some proposed paving projects have been such that departure from the standard proportions of concrete materials was imperative, from an economic stand-

point. Using the data derived from the study of the sand, or sands, involved and using either the method of design of concrete mixtures proposed by Talbot and Richart, or an amended form of it, a special mixture was designed for these cases. Success has attended this action in every instance. Experience with the mortar voids method of Talbot and Richart soon indicated the desirability of slight variations from the original method, at least for conditions encountered in Iowa.

This paper presents briefly the original method, calls attention to the variations as now practiced in Iowa by the author, and gives examples of the results obtained, both in the laboratory and in the field. For this presentation, the scope of the paper has been confined to a statement of experiences. Discussion of the laws of the composition of concrete as revealed by the work of Talbot and Richart, and and corroborated by the Research Division of the Iowa State Highway Commission have been deferred until opportunity permits the preparation of a more extensive presentation in which theoretical phases of the problem can be given more consideration.

The notations used by Talbot and Richart to represent each of the ingredients of the mortar and the concrete are used throughout this paper:

a = absolute volume of fine aggregate in a unit of volume of freshly placed concrete
b = absolute volume of coarse aggregate in a unit of volume of freshly placed concrete
c = absolute volume of cement in a unit of volume of freshly placed concrete
d = density or solidity ratio of the freshly placed concrete
v_m = voids (air and water) in a unit of volume of the mortar mixture of cement, fine aggregate, and water as it exists in the concrete
v = voids in a unit of volume of concrete. This, of course, will be equal to $1 - d$

It is evident that

$$a + b + c = d = 1 - v \qquad (1)$$

Since the mortar and the coarse aggregate together make up the unit of volume, it is also evident that

$$\frac{c + a}{1 - v_m} + b = 1 \qquad (2)$$

A third equation derived from equations (1) and (2) will be found useful:

$$v = v_m(1 - b) \text{ or } b = 1 - \frac{v}{v_m} \qquad (3)$$

The essential features of the original mortar voids method of comparing the concrete making ability of sands and selecting proportions of materials for concrete of a given strength, are the determination

of the basic water content of each of several mortar mixtures having different ratios of sand to cement; the determination of the mortar voids characteristics for the mortar at basic water content and for the same mortars having greater water contents, say 1.2, 1.4 and 1.6 basic water content, and the determination of the voids strength relationships upon specimens prepared from this series of mortars. These things having been done the selection of the proportion for a given set of conditions is merely a matter of arithmetical computation by means of simple equations derived from the mortar voids characteristics, for with normal coarse aggregates the strength of the mortar represents the strength of the concrete. A close approximation of the final values for the proportion can be made, after a little experience, without waiting for results of the strength tests. This is done by means of the Talbot and Richart formulas for strength:

$$(1) \quad S = \frac{32{,}000}{\left(1 + \frac{v}{c}\right)^{2.5}} \qquad \ldots\ldots (4)$$

$$\text{or} \quad (2) \quad S = 32000 \left(\frac{c}{v + c}\right)^{2.5} \qquad \ldots\ldots (5)$$

In these formulas the value of the constant will vary for different cements. For the particular cement or a group of cements having about the same strength, a comparatively small number of strength tests will give data for computing the value for the constant for the case at hand.

The first step in the evaluation of the concrete making ability of the sand is the study of its behavior in a series of mortars prepared with varying amounts of water. In this work, the ratios of sand to cement, and variation in water content, cover the entire range of combinations in which they will probably be used in concrete. Normally, the range for the sand-cement ratios will be found between values of $a/c + 1.0$, and $a/c = 5.0$; and the variations in water content from 1.0 basic to 1.6 basic water content.

The term, "basic water content," refers to that amount of water which is found to produce the maximum density of mortar, or, in other words, the minimum amount of voids in the mortar for a given sand-cement ratio. It is determined by mixing a given batch of sand and cement with varying amounts of water and computing the void space in a known volume of the mortar for each amount of water used. Usually this is done by adding to the sand and cement an amount of water well below the probable amount required at basic water content; removing from the mold; mixing with an additional increment of water, and again measuring the voids. Suc-

cessive increments of water are added until the point at which minimum void space is found has been passed. The amount of water required to produce a minimum value for the void space is called the "basic water content."

Notes of the typical procedure are shown in Table 1 and a graphical presentation of these and additional data for one series of proportions is shown in Fig. 1. The values for basic water content arranged for use in preliminary designs of the mixture and for comparisons of different sands are plotted as shown in Fig. 2. The more detailed final design requires the data shown in Table 2, which is also shown graphically in Fig. 3. These data were obtained from 2 x 4 in. mortar specimens. Each value except the strength values represent ten tests; the strength values are the average of five tests. Talbot and Richart base their proposed method of designing the mixture upon data taken from similar curves and from data shown graphically in Figs. 4, 5 and 6, which have been reproduced from data published in the Bulletin No. 137, previously referred to.

The curves shown in Fig. 4 and 6 are graphic presentations of data obtained from a study of three sands. In Fig. 4, it will be noted that the plotted values show cosiderable dispersion. The curve is drawn through values for specimens at basic water content. The wide variations from this curve, in general, represent higher water contents. Thus, for a given void-space ratio the range of strength values is rather wide in the territory of the commonly required strengths. Talbot and Richart determined the relation between the strength at basic water content and the strength at higher water contents for each of a series of values of the void-space ratio. This relationship, plotted as a percentage of the strength at basic water content against the relative water content produced the reduction factor curve shown in Fig. 6. "This reduction curve means that at a relative water content of, say 1.30, the strength of the concrete is 0.73 of that which would correspond to the same voids at basic water content." Fig. 5 shows the results of strength values for one sand computed to the values at basic water content. This device reduces the strength values to a narrow band and prepares the data for the method of design proposed by the authors.

Data similar to those shown in Figs. 3, 4, 5 and 6, together with the equations previously shown are used by them to design a concrete mixture as outlined below. The unit strength requirement at an age of 28 days being given; the mortar voids characteristics of the sand

TABLE 1—TYPICAL MORTAR VOIDS TEST

(*Tamped*)

Sand—Meadow, Nebr.—A-874
$A/C = 3.5$
$a^1 + c^1 = 203$ cc

Specific gravity of sand—2.622
Specific gravity of cement—3.15
Volume of mold—203 cc

Sand		Cement		Weight of Water Gm.	Total Weight of Mat'l	Weight Mortar in Batch	Vol. of Batch cc	Abs. Vol./Unit Vol.		
Weight Gm.	a^1 cc	Weight Gm.	c^1 cc					Density	Voids (v_m)	Water (w_m)
414.0	157.9	142.0	45.1	40	596	404	299	.678	.322	.134
				45	601	420	290	.700	.300	.155
				50	606	450	273	.743	.257	.183
(Basic Water Content)				55	611	463	268	.758	.242	.205
				60	616	459	272	.745	.255	.220
				65	621	459	274	.739	.261	.237
				70	626	457	279	.729	.271	.251
				80	636	452	286	.711	.289	.280

TABLE 2—MORTAR VOID STUDY—SAND INVESTIGATION

Sand No. A-874—Lyman-Richey Sand Co., Meadow, Nebraska—Tamped Age—7 and 28 Days

A/C	1			2			3.5			5		
Water Content	Basic	1.2	1.4	Basic	1.2	1.4	Basic	1.2	1.4	Basic	1.2	1.4
Absolute volume per unit of volume:												
Cement.........c	.328	.314	.296	.238	.229	.221	.167	.163	.156	.127	.124	.121
Sand...........s	.329	.314	.297	.478	.461	.443	.588	.572	.549	.637	.621	.605
Water..........w	.308	.351	.389	.247	.285	.309	.206	.238	.267	.188	.220	.250
Voids...........v	.343	.372	.407	.284	.310	.336	.245	.265	.295	.236	.255	.274
Density.........d	.657	.628	.593	.716	.690	.664	.755	.735	.705	.764	.745	.726
w/c	.939	1.12	1.31	1.04	1.24	1.40	1.23	1.46	1.71	1.48	1.77	2.07
v/c	1.04	1.18	1.38	1.19	1.35	1.52	1.47	1.63	1.89	1.86	2.06	2.26
Cu. ft. of water per bag of cement	.451	.536	.631	.498	.597	.671	.593	.700	.821	.710	.851	.990
Cu. ft. of voids per bag of cement	.502	.569	.660	.572	.650	.730	.705	.780	.908	.894	.986	1.085
Comp. str. in p.s.i.:												
7 days...............	5760	3646	2277	4230	2375	1610	2505	1451	814	1688	1078	845
28 days..............	8362	6094	4300	6695	4309	3039	4450	3118	1883	3128	2274	1605
Equivalent load at basic water content:												
7 days...............	....	....	....	....	....	....	....	....	....	....	....	....
28 days..............	....	....	....	....	....	....	....	....	....	....	....	....
Gallons of water per cu. ft. mortar...........	2.304	2.626	2.910	1.848	2.132	2.312	1.541	1.781	1.998	1.407	1.646	1.871

being determined; and a general strength-cement-space curve for similar sands being available, the designer next requires an assumption of the quantity of water required to give the desired workability to the concrete. This assumption having been made, the designer determines from the reduction factor curve the strength value at basic water content. He then obtains from the strength curve the value for the cement-space ratio which corresponds to that strength. Having the values for the cement-space ratio, the value for the sand-cement ratio is taken from the type of curve shown in the central part of Fig. 3. If the relative water content assumed was some other value than that for which curves are shown, the value for a/c can be obtained by interpolation. This value being determined, a value for

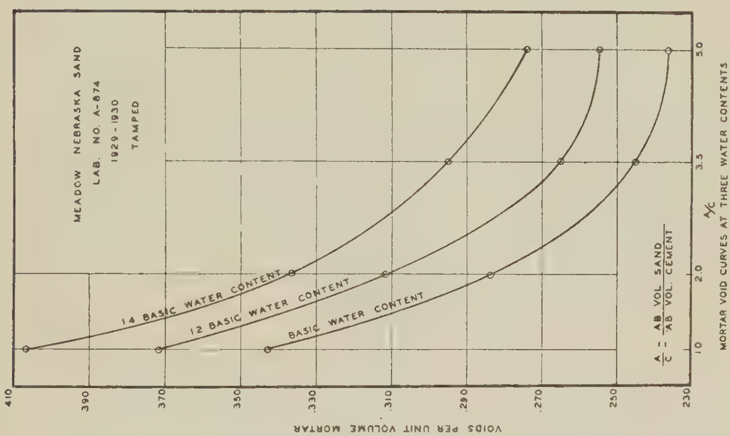

MEADOW NEBRASKA SAND
LAB. NO. A-874
1929-1930
TAMPED
1 4 BASIC WATER CONTENT
1 2 BASIC WATER CONTENT
BASIC WATER CONTENT
A/C = AB VOL SAND / AB VOL. CEMENT
A/C
MORTAR VOID CURVES AT THREE WATER CONTENTS
VOIDS PER UNIT VOLUME MORTAR

FIG. 2

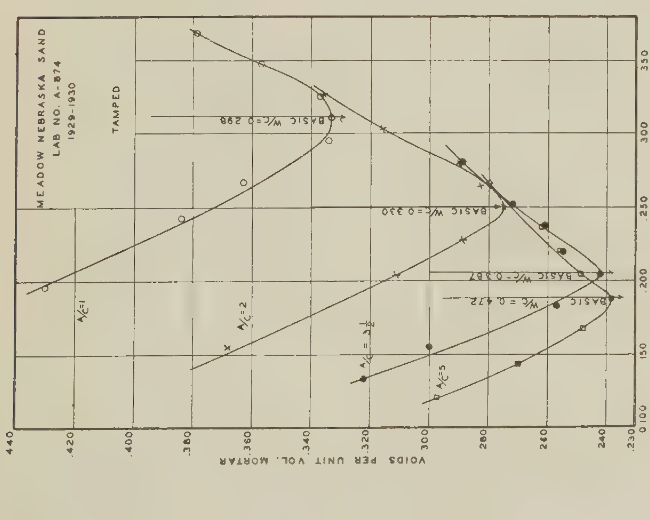

MEADOW NEBRASKA SAND
LAB NO. A-874
1929-1930
TAMPED
BASIC W/C = 0.298
BASIC W/C = 0.330
BASIC W/C = 0.387
BASIC W/C = 0.472
A/C = 1
A/C = 2
A/C = 3½
A/C = 5
WATER CONTENT PER UNIT VOL MORTAR
VOIDS PER UNIT VOL. MORTAR

FIG. 1

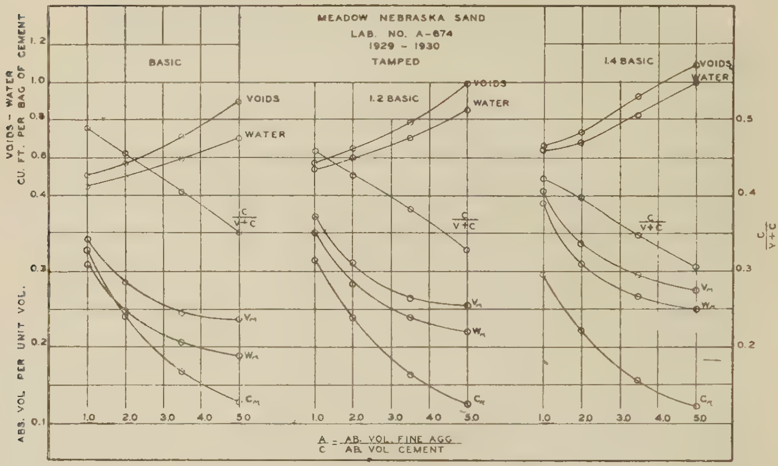


Fig. 3

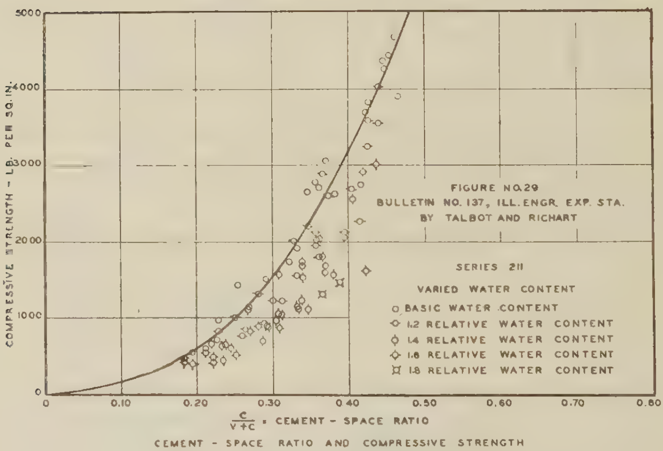
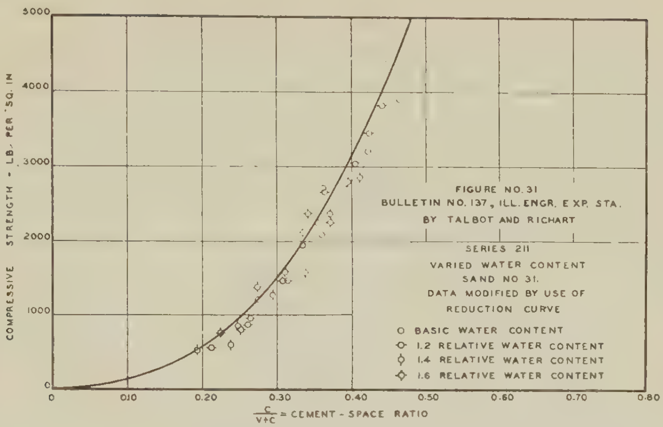


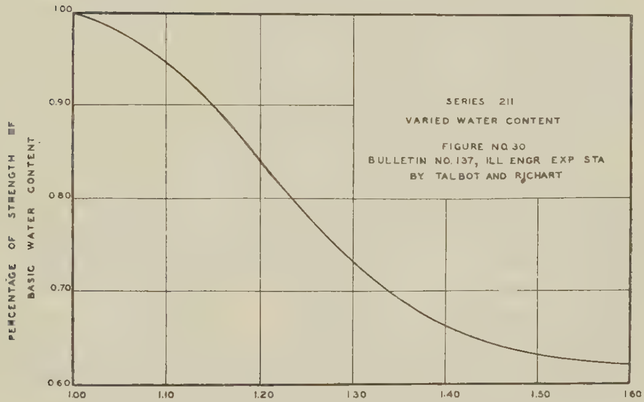
Fig. 4

b is assumed and the values for the absolute volumes of the remaining constituents in a unit volume of the concrete are computed. From these the proportion for the mixture can be stated either by absolute volumes, or by weight, as may be desired. The transition from the absolute volumes of the materials to their respective weights is readily accomplished.



CEMENT – SPACE RATIO AND COMPRESSIVE STRENGTH

Fig. 5



REDUCTION CURVE FOR CONCRETES OF VARIED WATER CONTENT

Fig. 6

During the last five years the relationships between cement, voids, strength of mortars and concrete have been extensively investigated by the Iowa State Highway Commission. The relationships established by the work of Talbot and Richart in their mortar-voids experiments have been found to exist for all sands available for use in Iowa. The author is in accord with the results insofar as basic laws of com-

position of mortars and concrete and the ralationship between their unit strength and their composition is concerned.

Brief experience with the tentative method of design using the reduction factor curve, however, soon indicated that some variation of the method of use of the data for designing proportions with Iowa sand would be necessary if concrete having the minimum departure from the desired strength were to be produced. It will be recalled that a general curve of strength values, and cement-space values was prepared for use in the Talbot and Richart proposed method of design, and that all strength values were computed to basic water content. Then this curve was used in designing, through the medium of a reduction factor curve, another general curve, that is, one generated by plotting values obtained for a number of sands. This is referred to at this time for emphasis, for it is at this point that the principal variations introduced by the author for use in Iowa have their beginning.

In the preparation of a general curve, the individuality of a given sand is merged with that of others. Apparently, in many instances, the use of a general curve introduces the use of characteristics sufficiently different from those of the individual sand to obliterate almost wholly its individuality for designing purposes. As an experiment the strength-void curves for each sand were plotted separately. As may be expected, a family of curves of the same general shape resulted. The analysis was carried further—a separate curve for each water content for each sand was prepared. This revealed striking differences in the characteristics of the different sands and indicated possible reasons why the tentative reduction factor curve would fail to be reliable at times.

In Fig. 7, the strength-void-cement relationships for mortar at the age of 28 days are shown for a sand typical of one of a related group of sands. For a sand of this type, with the curves for the different water contents separated, in the proper order and approximately parallel, the reduction factor curve seems applicable. For those sands for which these curves are practically coincident for a considerable portion of their lengths for all water contents the reduction factor curve fails to be applicable, particularly so when the curves fall on one side or the other, and at some distance from, a general curve for the group of sands for values at basic water content.

It was to eliminate this difficulty that some device other than the reduction factor curve was sought. If they may be judged by their results, two fairly successful devices were found, each using the strength curves generated for the individual sand. Later these de-

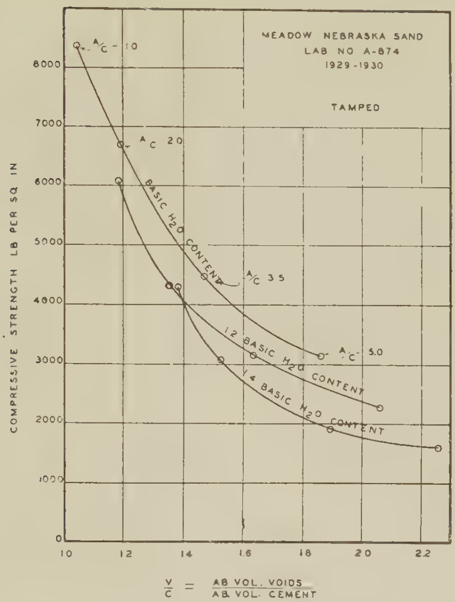
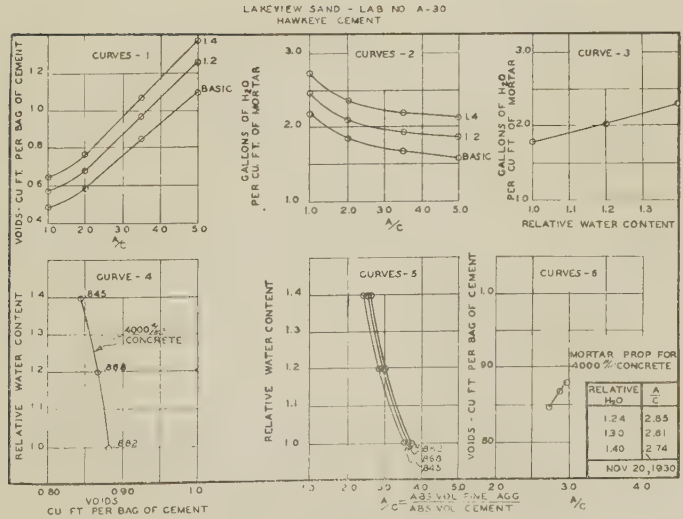


FIG. 7



RELATIVE H_2O	A/C
1.24	2.85
1.30	2.81
1.40	2.74

FIG. 8

vices were merged into the one now used, which will be described briefly.

In this method, the details of securing the mortar-voids data, preparing mortar strength specimens and assembling data are the same as developed by Talbot and Richart, except for minor improvements of technique which have provided for a greater uniformity of results. The data for each sand are studied and used separately. The work of Talbot and Richart establishes the general laws of the relationships of the cement, voids and strength of mortar and concrete. The individual sands have been found to be governed by these same laws, but may have a slightly different range of values. The plotted results for each sand indicate the family relationships, but express directly the individual characteristics of the sand.

Instead of preparing a general curve for several sands, the individual curves are used throughout. Further, it has seemed advisable to analyze these as shown in Fig. 7. With these curves the method of design begins. Given the required unit strength for the concrete at 28 days, the values for the void-cement ratio at each of the water contents intercepted by the chosen strength values are determined. These values are plotted as shown in curve 4 of Fig. 8, which, incidentally, is for a sand that fails to permit the use of the reduction factor curve. The $\frac{c}{v+c}$ curve may be used if desired, but the v/c curves are somewhat easier to prepare and are equally satisfactory. In general, the v/c relationship is expressed in terms of the cubic feet of voids per 94-lb. bag of cement. This permits use of values for direct checks upon computations at the completion of the design.

Curve 4, of Fig. 8 being plotted, attention is turned to the estimate of the quantity of water to be used. Here it is necessary to review experiences with similar classes of concrete. A brief computation reveals the fact that a cubic foot of mortar in similar concrete contained a certain number of gallons of water. The sand-cement ratio of that concrete is known and from voids tests upon concrete the rate of change of the water content with variations of the a/c ratio is known. The preliminary design, made before the mortar strength tests were available, has indicated the approximate value for the a/c in the new proportion. The gallons of water per cubic foot of mortar required for this proportion are then known.

The mortar voids data of the given sand for the water content are plotted as shown in curve 2 of Fig. 8, and the values for the water content of the old a/c are plotted as shown in Curve 3. Entering

curve 3 with the water requirement of the new proportion, the relative water content for the mortar to be used is discovered at the intersection with this curve. Curve 4 is entered with this value for relative water content and the voids for the new proportion are obtained. Interpolation curves 1, 5 and 6, now provide for the determination of the sand-cement ratio of the new proportion. This being determined, the values for the absolute volumes of the ingredients in the mortar are determined from curves such as those shown in the lower part of Fig. 3. Interpolation is required if the relative water content is other than that for which these latter curves are available. Experience with a similar class of concrete and the circumstances attending the design will indicate the ratio between the sand and coarse aggregate for the case at hand. By means of this ratio the value for the absolute volume of coarse aggregate to accompany each cubic foot of mortar is at once available. The absolute volume values of each of the materials is transformed to the weights of these materials and the design of the mixture is completed. Numerical illustration of this procedure was published in 1930.*

The design of concrete mixtures by this method to obtain a specified strength of concrete in both the laboratory and field concrete has been attended with a high degree of success. First experiences with the Talbot and Richart method and with the first amended form of that method were fairly successful. For example, mixtures designed for 42 representative sands produced within ten per cent of the strength of concrete desired for 50 per cent of the design. For the first amended form the design strength was 4000 p.s.i. at 28 days. The average of the 42 sands was 3892 p.s.i.; the average variation from 4000 p.s.i. was 14.7 per cent and the maximum variation was a −38.3 per cent. For the Talbot and Richart method the design strength was 4000 p.s.i., but for the same water content as for the above design, that is, 1.2 basic, from the reduction factor curve of the group, a strength value of 4760 p.s.i. at basic water content was used. The average strength was 4532 p.s.i. at 28 days; the average variation from 4000 lb. was 15.5 per cent and the maximum variation was a +44.6 per cent.

In either case the average of the values for the series is perhaps acceptable, but the individual construction projects are not built of a product which is composed of equal amounts of all of the sands available. The general accuracy of the design was established for fifty per cent of the cases, but the fifty per cent of misses presented the

*Proceedings of the Tenth Annual Meeting of the Highway Research Board, "The Design of a Concrete Mixture," by Bert Myers and Mark Morris.

real problem. Many of these were off but little; an extension of the requirement of ten per cent on either side of 4000 lb. to 12.5 per cent would have included the major portion of them.. A few, however, were apparently incorrigible when judged by the method of design used.

On these highly individual sands special attention was concentrated for another series of tests. Out of this work grew the method of design now used. This has been applied to a few important representatives of the group failing to produce satisfactory results for previous methods, and to representatives of those that did give satisfactory results by those methods. Success has attended both demonstrations in the laboratory and gave confidence for extending its use to field practice where needed.

During the progress of the work with the group of 42 sands, proportions for concrete were designed as rapidly as data from the mortar voids specimens were available. Concrete specimens were prepared with the same cement and the curing was done in the same moist storage room as was used for the mortar specimens. The success of these designs has been described. The work done to search out the causes of the failures was performed a year later with a different cement. However, in these specimens the cement and curing were the same for both the mortar and concrete specimens. The relationships for the mortar and concrete are the same as for the previous work, but a direct comparison of the cement content per cubic yard cannot be made on account of the slightly higher strengths obtained for the fresh cement. The only data in Table 3, which can be directly compared for the different proportions are those showing the percentage variation from the design strength, or 4000 p.s.i. at the age of 28 days. Other comparisons can be made by means of computations beyond the scope of this paper. The cement content gives the best idea of the variation in the proportions. In all cases the coarse aggregate is 1.5 times the amount of the fine aggregate.

The standard proportion is one of a group designed about 14 years ago. Each of the proportions of this group produces concrete of equivalent strength. The group provides for the use of combinations of fine and coarse aggregates in varying amounts such that the sand is from 33 to 100 per cent of the total aggregate. The standard proportion shown in the table is the second of the series. It provides for a relation of sand to gravel in ratio of 40 to 60 and a cement content of 1.71 bbl. of cement per cu. yd., when the water-cement ratio is 0.466 by weight, or 5.25 gal. per bag of cement.

TABLE 3—LABORATORY SPECIMENS SHOWING PERCENTAGE DEPARTURE OF AVERAGE STRENGTH VALUES FROM 4000 P.S.I.

Sand No.	Standard Proportion			Talbot and Richart Method Proportion			New Mortar Voids Method Proportion		
	% Variation	Cement Bbl./Cu. Yd.	Water Gals./Bag	% Variation	Cement Bbl./Cu. Yd.	Water Gal./Bag	% Variation	Cement Bbl./Cu. Yd.	Water Gal./Bag
4	+38.9	1.726	4.74	+24.1	1.620	5.03	—0.35	1.325	5.11
6	+23.6	1.720	5.02	+9.1	1.470	5.76	—2.50	1.318	5.98
36	+12.2	1.720	4.80	—8.2	1.306	5.98	+5.40	1.528	5.04

An inspection of the values for the variation from 4000 lb. concrete shown in this table indicates that the standard proportion which was designed for average conditions, several years previously, produces concrete much higher than 4000-lb. when these particular sands were used with the better cement of the year 1928. Further inspection reveals the fact that the method of designing proportions using the reduction factor curve fails to provide an accurate means of selecting the proportion which produces 4000-lb. concrete. Sand No. 36 performs fairly well with the Talbot-Richart method, Sand No. 6 does sometimes, and Sand No. 4 never does. These sands, therefore represent the range of variation found in Iowa.

The per cent variation from 4000-lb. concrete for the method of design using the mortar voids data as described in this paper is shown under the heading, "New Mortar Voids Method." The small variation indicates the success with which this method may be used for all of the sands falling within the range of characteristics exhibited by the representatives illustrated here.

Experiences in the field began before the mortar voids method of approach to the problem of designing was thoroughly understood. Enough data were available to indicate that advantages of the unusual concrete making ability of a particular sand might be taken if the need arose. Late in the season of 1927 it became necessary to consider, for economic reasons, a departure from the standard proportions for the construction of a series of paving projects in a section of the state where concrete aggregates were very scarce and where those available must be shipped long distances.

Preliminary studies indicated that the logical and readily available material was a river deposit in which about 15 per cent of the material as prepared for concrete was retained on a No. 4 sieve and 85 per cent passed this sieve. The sand portion had a rather coarse grading and an ability to make high strength concrete in the standard proportions. If used in this proportion, the ratio of the weight of

cement to the total weight of aggregate would be 1:2.36, for a material of this grading. The cement content would be 2.71 bbl. per cu. yd. of concrete and the compressive strength at 28 days something over 5,000 p.s.i.

Using the mortar voids data as a guide it was decided that all the saving demanded by the first project could be secured readily. This was done with a proportion of 1:3.5, by weight. The cement used in the laboratory indicated a strength of about 4600 p.s.i. at 28 days. The fresh cement used in the field gave, for the specimens made during construction of the project, an average strength of 5177 lb. at that age. The cement content was 2.10 bbl. per cu. yd. of concrete, or a saving of 0.61 bbl. per cu. yd.

Early in the season of 1928, additional data were available for further study of the concrete making ability of this material. For a group of projects awarded shortly after this time, the proportion was revised to 1:4.0. The cement used in the laboratory showed that a strength of about 4100 lb. might be expected. The specimens made in the field during construction had an average strength of 4639 p. s. i. The cement content was approximately 1.90 bbl. per cu. yd., a saving of 0.81 bbl. per cu. yd. when compared with the standard proportion. The saving per mile and for the year is shown in Table 4.

The mortar-voids studies were sufficiently advanced during the winter of 1928-29 to permit the full use of the Talbot and Richart proposed method of design for the 1929 construction season. By this method a proportion of 1:3.90, by weight, was approved with the expectation of obtaining concrete having a unit strength of 4000 lb. The cement content for this proportion is 1.90 bbl. per cu. yd. of concrete. This proportion has been used without change each season since that time. In 1929 the field specimens gave an average crushing strength of 4278 p.s.i. at 28 days, and in 1930, 4696 p.s.i. The cements used generally during 1931 gave higher early strengths than the cements used in previous years. This resulted in an increase in the unit strength at the age of 28 days for the field specimens made during that year's construction. The average of the specimens for the 1:3.90 proportion was 5108 p.s.i.

The saving of cement is obtained by comparing the amount used with that amount required by the original standard proportion which would have been used if researches in mortar voids had not demonstrated a more economical manner of use. Advantages of the later method of design could be applied to the 1:3.9 proportion to get a little closer to the 4000-lb. strength. This would reduce the cement slightly. To date it has not appeared advisable to do this.

TABLE 4—RESULTS OF COMPRESSION TESTS ON SPECIMENS MADE IN FIELD AND SAVING IN CEMENT DUE TO DESIGN MIXES

Year Built	Number of Projects	Total Length Miles	No. of Spec. Tested	28-Day Compr. Str. p.s.i.	Proportion By Weight	Saving in Cement Barrels
1928	1	5.189	23	5177	1 : 3.5	7,138
	6	57.052	351	4639	1 : 4.0	104,208
1929	17	159.629	1,068	4278	1 : 3.9	291,570
1930	18	154.721	1,131	4696	1 : 3.9	282,606
	2	14.300	120	4060	1 : 2.214 : 3.270	5,869
1931	7	34.327	279	5108	1 : 3.9	62,700
	4	40.490	270	4899	1 : 2.241 : 3.111	16,619
Totals	55	465.708	3,242	....	......	770,710

TABLE 5—CHARACTERISTICS OF CONCRETE FOR SPECIAL MIX

	Concrete Designed	Concrete Used
Water-cement ratio, lb. per lb.	0.447	0.445
Water-cement ratio, gal. per bag	5.04	5.01
Abs. Vol. cement per cu. ft. of concrete	0.1085	0.1092
Abs. Vol. water per cu. ft. of concrete	0.2918	0.2895
Abs. Vol. coarse agg. per cu. ft. of concrete	0.4378	0.4342
Abs. Vol. air voids per cu. ft. of concrete	0.0096	0.0143
Abs. Vol. solids per cu. ft. of concrete	0.8353	0.8329
Voids-cement ratio, cu. ft. per bag	0.730	0.733
Bbl. cement per cu. ft. of concrete	1.528	1.536

TABLE 6—RESULTS OF STRENGTH TESTS ON SPECIAL MIX

	Age Days	Compr. Str. p.s.i.
Design	28	4000
50 Specimens carefully made in the field by laboratory crew	28	4170
96 Specimens made in field by paving inspectors	28	4109
7 Cores taken from pavement	33—38	4243
45 Cores taken from pavement	40—75	4370

Early in 1930, further studies of the mortar-voids data resulted in the proposed improvement in the Talbot-Richart method of design. Opportunity to use this new method in the field came rather unexpectedly. It was proposed to place in competition with the 1:3.9 mix a special mix using the same fine aggregate and a limestone coarse aggregate. The fine aggregate to be credited with an average of 10 per cent of pebbles. The proportion of materials to produce 4000-lb. concrete was found to be 1:2.214: 3.270, by weight, after correction for grading and specific gravity of materials. For the 1931 work this proportion changed slightly as shown in Table 4 to effect the slight correction for yield indicated in the first year's use. Data relative to values for quantities of materials used by this design are shown in Table 5.

In Table 6 are data showing the success of this design in producing concrete of the desired strength. The successes in the laboratory and field with this design indicate that similar success may be expected with designs for which as yet opportunity has not been available for demonstrating their success other than in the laboratory.

The degree of success attained in the field and in the laboratory with the new method of design based on the study of the mortar voids characteristics of mortar further confirms the author in the belief that in the work of Talbot and Richart are to be found the basic principles underlying the rational design of concrete mixtures.

Readers are referred to the JOURNAL *for January 1933, for discussion which may develop. Such discussion should reach the Secretary by November 1, 1932.*

Discussion of a Paper by T. C. Powers:

"Studies of Workability of Concrete"*

Wallace F. Purrington† and *Harold C. Loring*†—Mr. Powers begins his summary and conclusions with this statement: "This paper reports a study of the workability of concrete mixtures by means of a new test, called the 'remolding test,' which measures the relative effort required to change a mass of concrete from one definite shape to another by means of jigging." We will attempt to show that in our opinion Mr. Powers has *not* measured workability but has measured mobility under the effect of dynamic forces.

The "remolding test" is a measurement of mobility, that is, the ease in which the individual grains of fine and coarse aggregate flow over each other in a cement and water matrix when subjected to jigging on a flow table.

The energy necessary to mix and place concrete may be measured. The practical method for doing this is, first, to determine the power output of the mixer taking into consideration the work accomplished by forcing the blades through the concrete. It is quite obvious that by changing the rate of speed the mixer turns or by changing the size of the blades that different values for power will be obtained. Let us carry this thought still further by supposing that this concrete is dumped on the ground at Point A which by hand labor must be placed at Point B. The laborer will be forced to expend a certain amount of energy to do this. Assume, first, that the laborer moves this material at a slow rate of speed. The study of this operation then involves three elements—mass, space and time. The terms ordinarily used for measurement are foot pounds per second. If any one of these elements be changed, the result will, of course, vary. If concrete is easily handled, that is, if it is mixed and placed with the minimum of effort then the material is said to be "workable." If this effort is great it is said that the material is workable with difficulty. From this postulate, we state that there are two underlying principles of workability which must be considered.

1. The resistance of the mass to a force.
2. The rate of application of a force.

*A. C. I. Journal, Feb. 1932; *Proceedings*, Vol. 28, p. 419. The discussion is continued from June, Vol. 28, p. 693.

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We are convinced from our work on this subject [1 and 2] that the above basic factors *must be measured* in order to determine the workability of concrete mixtures. From our work on the design of concrete mixes using the water-cement ratio law, we have found that there is a definite relation between this law and the workability of concrete. On page 659, Fig. 3[2], we show the relation of 2000-3000-4000-lb. concrete to the net power consumption at various speeds at the same relative consistency. From these results we can infer that there is less energy required to overcome the resistance of the mass in a 3000 lb. concrete than that required in 2000 or 4000 lb. concrete, consistency being constant. The reason for this difference is easily explainable. The 2000 lb. concrete lacks proper lubrication with the result that, while being mixed, there is a high friction of the under-lubricated grains. The 4000 lb. concrete has a high degree of plasticity but there is also a high degree of cohesiveness due to the greater quantity of cement. In consequence, the energy necessary to cause shearing action is very high.

There can be no question that the 4000 lb. concrete would be favorable when mobility was considered. Based on *mobility alone* the 4000-lb. concrete would be considered more workable than the 2000 or 3000-lb. concrete. However, if the choice of moving either of the concretes was left to the laborer, the 4000-lb. concrete would not be chosen. Yet, under the "remolding test" it would probably require the least jigs to deform. Based on this test it would be considered to have the greater "workability."

An illustration of the forces at work when workability and mobility are considered can be taken from a simple experiment which can be carried out on certain types of soils. A quantity of very fine sand is mixed with enough water to mold into a small cylinder. This is placed on a glass plate and then lightly tapped. Immediately we see this material flatten out, the water coming to the top of the pat. In this case we have a dynamic force in operation. When this cylinder is put on the plate and pressed by the thumb we notice that the water disappears with a consequent change in volume. When the pressure is increased, shearing action takes place. In this case a static force is operating. The amount of work required to deform the sample a given amount is much greater if static pressure is applied than if the sample is subjected to shaking or light jigging. In the first case, the coarse constituents brace themselves against relative displacement. In the second case, the opposite is the case. A loose sand cannot be made dense by simple pressure but easily so by shaking or rodding.

[1] "The Determination of Workability of Concrete," A. S. T. M. Proc. 1928, Vol. 28.
[2] "Further Studies on the Workability of Concrete," A. S. T. M. Proc. 1930 Vol. 30, Part II.

Workability of a mass is its resistance to static deformation and not its mobility under the effect of jigging Both have little in common.

The "remolding test" is nothing more nor less than a refined method of conducting the flow table test. It does not *measure* resistance to *remolding*, neither does it take into consideration the rate of application of a force.

*G. M. Williams**—In general the writer agrees with the author's definition of workability which involves the two important factors, ease of flow and resistance to segregation. While workability or placeability is dependent mainly upon mobility or flowability of the mass it also depends to some extent upon the ability of the mortar to prevent segregation of the larger aggregate particles during the process of transporting and placing the concrete. Flowability is always an essential factor and must be secured if placement is to be made with a reasonable amount of effort. Segregation tends to increase the amount of effort somewhat but its main effect is to result in a non-homogeneous mass with consequent variations in strength and permeability. Excess flowability partially counteracts segregation in relation to energy required for placement but freedom from segregation is to a much smaller degree a substitute for flowability. Mobility or flowability of the mass is dependent mainly upon the wetness or fluidity of the mortar while segregation is dependent upon adhesiveness and relative quantity of the mortar. In general in a given mix an increase in flowability increases quantity of mortar but decreases adhesiveness and the resultant effect is to increase segregation. While adhesiveness is primarily due to the cement content of the mix, the fine aggregate assists in bulking or swelling the cement paste thereby increasing the total volume. Segregation is usually evidenced by separation of coarse aggregate particles from mortars which may have good adhesive properties but are present in too small relative proportions, or by the flowing away from the aggregate of soupy fluid mortars which have little or no adhesive properties owing to high water and low cement contents. The first condition can generally be corrected by the employment of a higher sand-gravel ratio, or when practicable, substitution of a larger maximum size aggregate which will furnish fewer aggregate particles to be held in suspension. Mixtures containing fluid mortars should never be permitted in practice. Segregation in such cases can be eliminated by the use of richer mixes or change in aggregate grading.

Undoubtedly more attention to workability in the past would have added years of life to concrete structures which are now showing

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signs of disintegration due to weathering. Mixtures which were inherently harsh and unworkable with fluid mortars due to low sand content such as 1:3:6 and 1:2:4 were made worse by the use of still more water to facilitate chuting and to assist in obtaining smooth surfaces with a minimum of labor. Under such conditions a satisfactory surface appearance is no indication of uniformity in the interior of the mass. For the same cement content a non-segregating, workable mix will usually have greatest strength and lowest permeability, two properties most essential to resist weathering.

To measure workability or the relative workability of concretes it would seem desirable to be able to measure or evaluate separately the mobility or flowability of a mix and the tendency of aggregate particles to segregate, since it is generally agreed that workability of structural concretes depends principally upon these two factors. Unless they can be separately measured the workability figure obtained is then the summation of the two variables and the relative effect of each one singly is unknown. The writer has employed both the slump test and the flow table for the measure of mobility and flowability since both methods have been in use and has found no evidence to alter his opinion, early arrived at, that the slump test is too erratic and unreliable for consideration in laboratory studies of concretes. In the field where aggregate proportions and gradings are practically constant the slump test serves as a satisfactory measure and control of consistency. In the laboratory where concretes varying widely in cement content, aggregate proportions and gradings are being compared, the slump test fails to furnish an accurate measure of mobility. The writer considers that the flow table method simulates the flowing and placing of concrete in practice and gives the truer measure of mobility in addition to more concordant results in repeated tests of the same concrete.

The writer believes that the author is on the right track in attempting to measure workability by means of the flow table principle. However, a reading of the paper, and particularly Appendix A, created the impression that the "remolding test" is a form of hurdle race to which the mixtures are subjected and that the values obtained are not measures of workability or even relative workability, but rather mobility or flowability figures for different conditions of restraint. This impression seems to be confirmed by the author's statement that, "It is not suggested however that this method provides a direct measure of workability as that term was defined in the paper, because it offers little indication of the properties of a mixture other than its mobility under a given condition." Reference to diagrams A and B

of Fig. 11 shows how greatly the workability figures change for a slight change in clearance of the apparatus, and also that these changes are not proportional for the three concretes involved. These curves show that a 1:3:6 concrete with a 2⅞ in. clearance is as workable as a 1:2:4 concrete with a 2⅝ in. clearance; also that a 1:2:4 concrete, in the intermediate consistencies, with the larger clearance is as workable as is the 1:1½:3 mix with the smaller clearance. Whatever the clearance, the inherent tendency of each concrete to segregate remains unchanged. Variation in clearance, as these results show, greatly affects flowability or spread of the mass but the comparative "remolding efforts" bear no relation to the homogeneity of the masses before or after the operation is completed, even though the clearance difference is very small.

The writer feels that Fig. 12 is rather unfair to the flow table and seems to indicate, on the part of the author, lack of experience in the use of this device or a disregard of data which the flow table makes available. The author compares remolding effort with mobility or flow of the mass as measured by the flow table. This evidence is good so far as it goes, but it is incomplete in that segregation, which must have been present for the concretes of low sand content, is not stated. So far as the writer is aware it has never been claimed that flow, as indicated by spread on the flow table is a measure of workability. However, in the early experimental work connected with the development of the flow table it was apparent that harsh or less workable mixtures furnished evidence of this characteristic by segregation of the aggregate particles from the mass when flowability was made constant. During the past year the writer presented a paper[1] in which the flow table was used to measure the change in workability, as measured by segregation, brought about by the addition of different admixtures to a segregating concrete. The concretes were brought to the same flowability as measured by the spread of the mortar line and the weight of the aggregate found beyond this mortar line used to express the relative workability. This same method applied to concretes similar to those in Fig. 11 of the author's paper clearly distinguishes between these concretes for differences in flowability ranging from near zero flow up to the point where the mortar becomes liquid and has lost its adhesive property.

In Fig. 1 curves A, B and C represent the segregation obtained with three concretes differing only in cement and water content. The decrease in segregation with increase in consistency found for the

[1] A. C. I. JOURNAL—February 1931; Discussion May 1931; *Proceedings* Vol. 28, p. 647 and 1133.

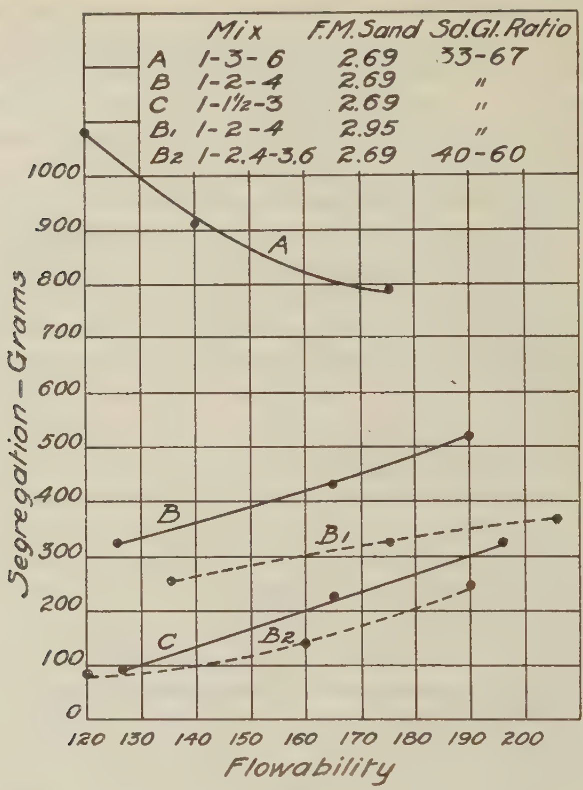

FIG. 1

1:3:6 proportion is typical of very lean mixtures. The effect of change in fineness of fine aggregate, or of variation of relative proportions of fine and coarse aggregates is shown by comparison of B, B_1, and B_2, all of the same cement content. Concrete B_1 contains a finer sand than B and an appreciable change in segregation is noted. In the concrete of B_2 the sand-gravel ratio was made 40:60 instead of 33:67 as for B and segregation becomes less than that which was found for the 1:2 ½:3 mix. Another relation, not illustrated here, for concretes of the same cement content is the decrease in segregation obtained by increasing the maximum size of aggregate particle, thereby reducing the total number of coarse aggregate particles in the mass. It should be noted that the decrease in segregation obtained for B_2 and B_1 as compared with B is due primarily to the greater paste volume which is in turn less fluid and more adhesive.

The few results shown are representative of data which may be obtained by using the flow table without additional apparatus. Accurate measurement is made both of flowability and segregation. It is of course essential, as the author points out, to avoid segregation of material in all steps involving sampling or molding. As harshness increases it becomes more difficult to obtain closely concordant results in repeat tests. However slight changes in either quality or quantity of mortar are reflected provided the mixtures show any segregation. The question may arise as to the relative workability of two concretes differing in one or more of the variable factors, cement content, aggregate grading or maximum size of aggregate but which may have the same flowability and show no segregation. Have these mixtures the same workability? Such a decision is beyond the limit of the flow table but the writer believes that workability will be the same in mass work or where reinforcing rods are not closely spaced, but if the material must be placed through a network of closely spaced rods the workabilities are not the same. It must be recognized that probably few structural concretes can be called workable, as defined previously, if the concrete must be sieved through reinforcing rods spaced from 1 to 2 inches apart as is necessary in some structural members.

*Jean H. Knox**—Mr. Powers has aptly illustrated the many steps by laboratory "trial method" necessary to arrive at comparable mixes to assure a homogeneous, workable mass of plastic concrete.

To do this, he has chosen fine and coarse aggregates whose uniformity of shape and surface texture furnishes ideal physical characteristics for experimental purposes.

These conditions rarely occur in actual work where aggregates of widely varying grading and characteristics must be considered.

In manufacturing concrete, strength is a secondary consideration and uniformity, homogeneity, impermeability, and durability are of primary importance.

Workability of concrete is a function of surface texture, areas, shape and size of particle, range of sizes, uniformity of grading, and properly proportioned parts to develop requirements in concrete by considering the aggregate characteristics rather than at first the cement or cement paste.

Feeling our way by original trial proportions may prove an endless task, therefore some methodical outline of measuring values of aggregate characteristics becomes necessary in commercial practice.

*Engineer Consultant, Dallas, Texas.

By using modified forms of Fuller's density curves as distribution curves, considering the combined aggregates excepting cement and water, a standard is obtained by which the number of trial proportion batches may be reduced to a minimum.

Mr. Powers is correct in asserting that reasonable density in combined aggregates is desired rather than maximum density—according to present practice of measuring density by weight.

Voids in the separate aggregates have little bearing on resulting voids and spacing of voids in the final combined mass.

Workability and strength are two factors necessary to quality of concrete but do not necessarily guarantee uniformity, impermeability, and durability.

Many of the present methods for concrete design seem to place emphasis on the least factor, cement, which is highly standardized, then assume conditions for the important factor, aggregates, whose variable characteristics are almost unknown or at least not given proper consideration.

Mr. Powers has chosen a middle ground, selecting materials of known favorable characteristics, then determining a suitable range of proportions for an easy identification of parallel degrees of workability.

In practice an engineer must produce a durable, economical structure with the materials available.

*C. D. Brown**—Mr. Powers approaches his subject as an economist. He sought the gradings and combinations of materials which require the least quantity of paste of a given quality and consistency to produce a given degree of workability. He shows forcefully that type, gradings, and combinations of aggregates, and consistencies and quantities of pastes are all related to each other and to the property of workability.

The theories developed by Mr. Powers are very probably correct, although minor exceptions may develop. They will prove valuable because they have connected the properties of concrete mixtures with the fundamental physical laws, and the concrete engineers of the country are not faced with another arbitrary and assumed scheme of factors, coefficients, or moduli to worry with for a while and then abandon.

Mr. Powers definition of paste, (p. 424) brings up the thought as to just what materials compose the paste. He speaks of the cement as being particles of solids, also he considers the solid particles of Celite

*Concrete Engineer, Jean H. Knox, Dallas, Texas.

as entering into the paste composition. Now, just what portion of the sand goes into the composition of the paste?

From the description given of the test, it seems that Mr. Powers has determined the specific viscosities of various pastes although he has not put the data into such terms.

From the gist of the context of the report, Mr. Powers seems not entirely satisfied with the results of the tests on variations in the grading of the coarse aggregate. He does find, however, that the amount of sand (the investigations have not yet extended to variations in the sand grading), amount of pea gravel and the grading of the coarse aggregate have a very marked relation with the minimum amount of paste it is permissible to use.

Throughout the paper, the fact keeps coming to the surface that the proportions and gradings of the various aggregates are prime factors for determining quantity and quality of paste for mobility.

It is regretted that, in dealing with variations in coarse aggregate grading, the combined gradings of fine and coarse aggregates were not kept forward. Not that any artificial grading should be sought, but that these combination gradings could be readily identified with the mobility produced.

A study of the gradings of the various combinations used in these tests proves very interesting. Lack of space prevents printing the plotted grading curves here, but the writer will be glad to send blue prints of these curves to those interested.

The first thing noticed is the wide range of possible gradings covered. No two of the combinations are alike, and they are all pretty well varied.

If the gradings of the combined aggregates are plotted and compared with smooth and uniform gradation curves, it is seen that the optimum sand content with each gravel grading produces a combined grading curve which is as smooth and uniform as is possible to produce with those materials. In other words, the most economical combination of the sand with each grading of gravel used in these tests is that combination in which the sand grading more nearly matches the gravel grading to give a smooth and uniform curve.

When the combination gradings of the gravels of each section with their optimum sand are examined, another relation is found. In each section, the combination grading plotting the most nearly along a smooth curve requires the least amount of paste for any degreee of mobility required. Those combination gradings giving the greatest variation from a smooth curve require the most paste.

When the gradings of all the combinations of each gravel with the optimum of sand are compared, it is found that some follow a smooth curve closely while others vary. Those which have a uniform grading require the least paste and are the most economical.

As Mr. Powers observed, the best percentage of sand in the combinations of sand and gravel is not necessarily the one producing the least voids in the combined aggregate, although the margin between the two values is usually small. However, the results of these tests indicate that the nearer the grading of the combined aggregate follows a smooth and regular curve, the more economical the concrete design will be.

*Ira L. Collier**—The writer has been asked, at various times, to make studies of the effect of admixtures on the permeability of concrete. Almost invariably the results have been the same, the plain concrete under laboratory conditions being as watertight as that containing the admixture. Yet we know that concrete placed in a wall with little or no inspection, containing an admixture, will in all probability have fewer gravel pockets, less evidence of segregation, and will give a wall more resistant to the percolation of water than one containing no admixture.

The writer has been developing for the last two years a piece of apparatus that would prove the above statement. Hence the problem has resolved into one of measuring the workability, or, to be more exact, to secure an index of the workability of concrete rather than measuring its permeability.

The apparatus as shown in Fig. 1 is the fifth and last with which experiments have been made. A duplicate of my third piece of apparatus was sent, early in 1931, to the Portland Cement Association with which to experiment. Fig. 9D of Mr. Powers' article gives an idea as to the appearance of the piece of apparatus sent them.

The results obtained, in most of the series, have been satisfactory and, in a measure, substantiate the conclusions found by Mr. Powers. The operation of the equipment is quite simple. Concrete is placed on a cover over the center cylinder A and scraped into the space B between the inner and outer cylinder. This operation is repeated until the space is filled. The cover is then removed exposing the opening C in the bottom. The handle of the flow table is turned at a uniform rate and the number of jolts required to bring the concrete to the opening C is considered an index of the workability of the concrete. The height of drop of flow table is ½ in.; 200 jolts is the maximum number recorded, for a very harsh mix, and 3 the minimum for a

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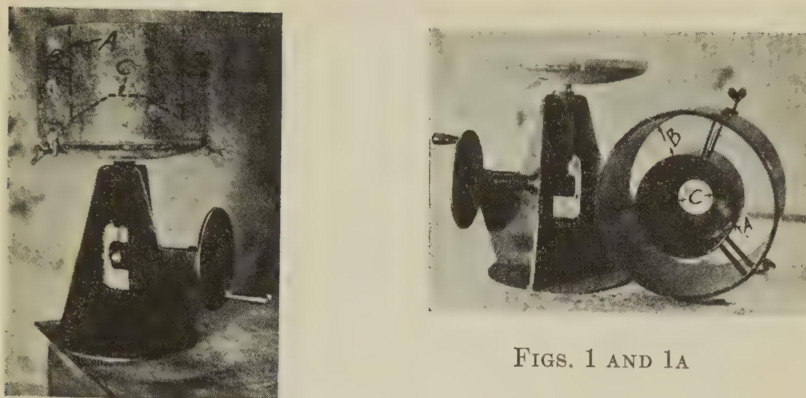

FIGS. 1 AND 1A

wet mix, one rich in cement and having a high percentage of fines in the sand.

The time the writer has for research is limited and the number of experiments few, to prove the value of this equipment when compared to the extensive tests conducted by Mr. Powers.

These diagrams confirm the statement of Mr. Powers that an admixture is chiefly beneficial in the under sanded mixes or the mixes containing few fines and in the lean mixes. They also indicate that with good design, that is, with optimum amounts of cement, sand, and fines in the sand, that a concrete can be made and placed with a minimum amount of effort.

The whole subject of proper design of concrete, for durability, workability and strength, and strength seems to be taking a secondary place, is becoming more complex as the results of each new series of experiments are published. It is only the larger organizations, such as cities, railways, state highway departments, or those placing large masses of concrete, that can afford the required laboratory or an engineer who is fully qualified to take advantage of all the latest developments as published in the many technical periodicals. As has been pointed out by other writers, the decision to use or not to use admixture depends upon the economical considerations involved. In many cases where the optimum amounts of materials are not available an admixture will prove valuable. Or where the necessary technical skill is unobtainable to determine whether or not the optimum amounts of materials are available, the use of an admixture may prove to be valuable.

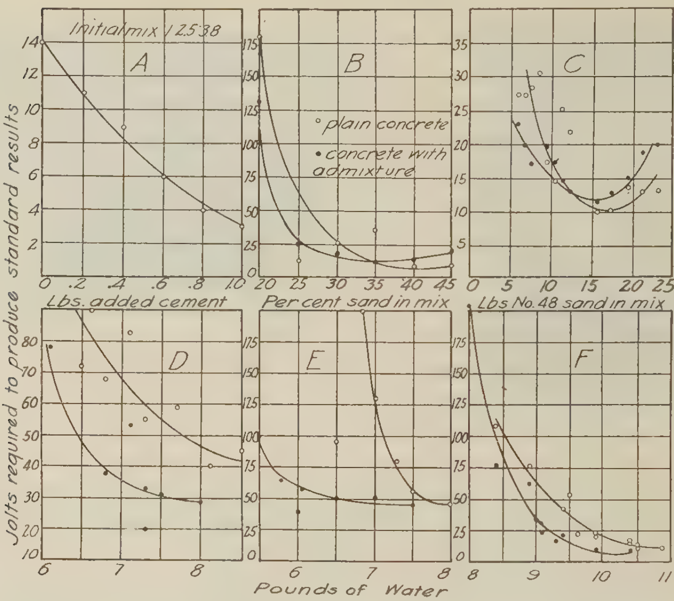

Fig. 2

A. Initial mix. 13.5 lb. cement; 33.9 lb. sand; 50.2 lb. gravel; 8.5 lb. water. Slump 8 in. Sand 6 lb. caught on No. 8; 6 lbs. on No. 14; 6 lb. on No. 28 and 15.9 lb. passing No. 28. Gravel 14 lb. passing 1½ in. square openings, none passing 1 in.; 16 lb. passing ¾ in. and 20.2 lb. passing ½ in. The maximum slump was 9 in. for the batch containing 0.8 lb. extra cement.

B Mix. 13.5 lb. cement; 84.1 lb. combined sand and gravel; 8.5 lb. water for plain cements and 7.2 lb. water for concrete containing Vaso admixture. Same grading of sand and gravel as given above under *A*. 17 c. c. of admixture added.

C Mix. 13.5 lb. cement; 33.9 lb. sand; 50.2 lb. gravel; 8.5 lb. water plain concrete; 7.2 lb. water for concrete containing Vaso. Sand contained an equal amount of material caught on No. 8, 14 and 28. 17 c.c. admixture. Gravel same as others.

D Mix. 13.5 lb. cement; 33.9 lbs. sand; 50.2 lb. gravel. Water variable. Sand, 9.3 lb. caught on No. 8; 9.3 lb. on 14; 9.3 lb. on 28; 6 lb. passing 28. Gravel same as others. 17 c.c. admixture.

E Mix. 10 lb. cement; 33.9 lb. sand; 50.2 lb: gravel; water variable. Same grading sand and gravel as in *E*. 17 c.c. admixture.

F Mix. 16.9 lb. cement; 42.3 lb. sand; 67.7 lb. gravel. Sand 5 lb. caught on No. 8; 5 lb. on 14; 10 lb. on 28; and 22.3 lb. passing No. 28. Gravel 10 lb. on 1½ in.; 15 lb. on 1 in.; 20 lb. on ¾ in. and 22.7 lb. on ½ in.. 17 c.c. admixture.

*Warren C. Bruce**—Further effort should be made to duplicate field conditions in the laboratory. If the author's method were entirely satisfactory from the laboratory standpoint, it still could not be used to predict accurately how a particular mix would perform in the field. Success will only come by first developing a large scale ap-

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paratus on the job and then after this job apparatus is perfected, reducing it to laboratory proportions.

There are a number of ideas in this connection, but to conserve space here I refer those interested to the writer's "Workability Found Necessary for Paving Concrete"[1]; "Studies of Paving Concrete," by Jackson Kellerman.[2] The latter article is one of the few successful attempts to correlate laboratory and field work on a sufficiently detailed scale to get at all the numerous variables.

Mr. Powers mentions that surface tension is a factor in workability, but discusses it only from the physical standpoint, and states that the nature of the material does not affect the workability. Has the author considered the fact that the nature of the material affects surface tension and consequently affects workability? When cement is mixed with water, chemical reactions begin which continue during mixing, placing and the life of the concrete. These chemical reactions affect the surface tension and other physical properties which in turn affect the workability. Concrete being tested for workability is continually changing its chemical and physical properties; consequently the time element should be carefully watched in comparing laboratory batches and especially when extending conclusions from the laboratory to the field. An attempt should be made to record the temperature within the mass of the concrete during workability tests. The mixing time used by the author was two minutes, whereas in field practice the mixing time is seldom more than one and one half minutes.

Rate of absorption of various cements and aggregates and rate of change of absorption with temperature affect the various chemical and physical properties of concrete.*

Heat diffusion in small masses of laboratory concrete is rapid; in larger masses on the job less so. This also affects the chemical and physical reactions. A fundamental law of physical chemistry states that the speed of a chemical reaction approximately doubles with each ten degree rise in temperature.

The author finds it convenient to consider the cement-water paste as a continuous body, and while this apparently simplifies matters it may not be advisable, since the study of workability involves a study of segregation, and in concrete we not only have the aggregate segregating from the cement-water paste but we have cement segregat-

[1]*Concrete*, Jan., 1932, p. 25.
[2]*Public Roads*, Aug., 1931.
*The author eliminated the factor of aggregate absorption by soaking the aggregate the 24 hours preceding the test. This helps get consistent laboratory results, but is one reason the remolding test will not accurately predict field results.

ing from the water. Thus the cement-water paste cannot be considered a continuous body except under those ideal conditions rarely met.

Most studies of concrete in this country have been from the physical and engineering standpoint and great progress has been made; we must expect further progress from the standpoint of physical chemistry.

*R. B. Young**—It is always a satisfaction to find someone holding the same viewpoint as yourself, especially when it has been arrived at by entirely different and independent means. For many years, as a member of the staff of the Hydro Electric Power Commission of Ontario, I have been studying the properties of concrete mixtures with a view particularly of developing the most economical mixtures for a given material, job and condition of placing. Our experience in this work bears out several of Mr. Powers' observations, one of which I think is of sufficient interest to mention briefly.

Mr. Powers brings out the point that if you add the proper or optimum amount of sand for the particular aggregate in question you have, for mixtures of similar workability, a cement content that is fairly uniform for a wide variety of materials. Our experience and studies agree with this conclusion. A number of years ago we carried out an extensive series of tests to determine what amounts of cement and aggregate were required with different kinds and gradings of aggregate to obtain concretes of a given compressive strength and workability. Workability to us meant ease of handling, placing and finishing, and lacking a satisfactory means of measuring this property, we made all the mixtures as nearly alike as could be judged by an experienced man from the way they mixed and handled. The method, while probably not scientific enough to satisfy the ultra-technical, proved very satisfactory and agreed with practical experience. These studies included several different gradings of sand and gravel and consistencies.

As a result of these tests we came to the conclusion that combining a sand and coarse aggregate in the proper ratio produces a concrete mixture of a given workability and strength therefrom with about the same cement content regardless of the grading of the aggregate used. Of course, such a broad conclusion is subject to modification and in this case we were only considering those gradings of aggregate lying within or just outside standard specification requirements such as one commonly meets with in practice. Also the conclusion cannot be applied to aggregates of different maximum size or of different minerological types, although even here the differences were usually

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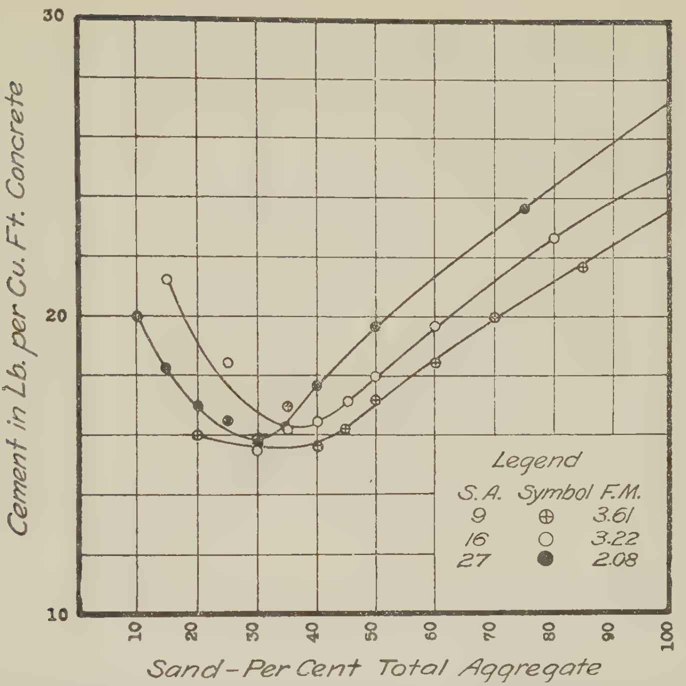

FIG. 1—RELATION BETWEEN SAND CONTENT AND CEMENT REQUIRED TO PRODUCE CONCRETE OF SIMILAR WORKABILITY TESTING 2500 P.S.I. AT 28 DAYS

not great. As will be seen from Fig. 1, the curves we obtained bear a marked similarity to those of Mr. Powers at the bottom of page 427, and while this may be a coincidence I am inclined to take it as further evidence that Mr. Powers' apparatus is giving a true picture of the workability of concrete mixtures.

AUTHOR'S CLOSURE

T. C. Powers—The discussions of the writer's paper are for convenience considered under two heads: Discussions pertaining to methods of tests; and those pertaining to theories, experimental results and conclusions.

DISCUSSION OF TEST METHODS

Exception was taken to the author's statement of the lack of a suitable laboratory measure of workability; yet no one has pointed out a thoroughly satisfactory device. The "remolding test" itself is not entirely satisfactory but it is believed to be a decided improvement over any others the author has seen. Professor Williams believes

that the ordinary flow test, together with determination of segregation is enough. The measurement of the segregation during the flow test helps evaluate the workability of a mixture, although some types of mixes which may not segregate on the flow table—the dry ones for example—may segregate badly in practice. That the standard flow test gives a true indication of mobility is open to question however. In fact, Fig. 12 of the paper and Mr. Smith's Fig. 3A definitely prove that the flow test will not give a true indication of the mobility of a mixture *under the conditions of flow imposed by the remolding test apparatus.* It follows, therefore, that if the conditions of the remolding test are comparable with those of a particular job, the flow test would not give a true indication of the mobility of a mixture on that job. Which of the two tests imposes conditions more nearly comparable with those on the job must be decided by judgment and experience. The writer believes that there are few conditions of field placement comparable with flow test conditions except at some intermediate stages in the process, such as movement down a chute.

As brought out in the paper, the flow test allows the concrete to spread in an unrestricted manner and thus gives an indication of the ease with which the mixture may be scattered, rather than brought to a definite, desired shape. Since Professor Williams states in his final paragraph that the indications of the flow test cannot be expected to apply to small or heavily reinforced sections, it is difficult to see how he escapes the conclusion that what he calls the "hurdle race" to which the mixtures are subjected by the remolding test is an important improvement over the older flow test, particularly as the "hurdle" can be adjusted at will to eliminate from the field "runners" of limited ability.

Tests have been made with the remolding test apparatus without the use of the inner ring, in which the harsher mixes flowed more readily, and results were more like those of the flow test. But there was still a distinct difference due to the necessity that the concrete in the remolding test finally attain a definite form and volume. Still other tests have been made in which neither rider nor ring was used. This condition was a still closer approach to the conditions of the flow test and harsh mixes tended to behave more like smooth ones.

Our first work in this study of workability began in August, 1930, with an apparatus which involved the remolding principle but which did not provide means of restricting the flow; that is, there was no inner-ring. Early in 1931, after seeing Professor Collier's apparatus and a sketch of one developed by Professor McNown of the University

of Kansas, Mr. McMillan suggested the use of a ring to restrict the flow of the concrete during the remolding test. We are in this way indebted to Professors Collier and McNown for the idea back of this part of the apparatus. However, the superficial resemblance between our apparatus as shown in Fig. 9D and that sent to us by Professor Collier is much more apparent than real. Professor Collier's apparatus required flow under restricted conditions but it did not involve the remolding principle.

Mr. Tucker refers to Fig. 11A as supporting his belief that the slump test may give equal or superior indications of workability. It seems that his statement regarding the relative mobilities which the 1:3:6 and the 1:1½:3 mixes show at a 1-in. slump should have been placed in quotation marks, for the exact words which he has used will be found in the last sentence of the second paragraph on page 445. In view of this we certainly cannot take exception to Mr. Tucker's statement. It is the basis of the remarks under the subhead "Limitations of the Remolding Test."

Regarding the behavior of the 1:3:6 mixture at different slumps as shown in Fig. 11A, Mr. Tucker has apparently overlooked the fact that a ring clearance of 2⅝-in. was used, comparable (p. 446) to a very severe placing condition. Under such conditions, a 1:3:6 mixture of the materials used would be difficult to place with any amount of water in the range shown. Increasing the water content above that giving about a 2-in. slump caused excessive segregation and particle interference, actually making the mixture more difficult to place. The slump test, however, taken at its face value, shows that increasing the water in the 1:3:6 mixture would greatly improve its mobility. Which of these indications is correct must be judged according to individual experience.

Fig. 11 and 12 were offered as proof that the remolding test distinguishes between mixtures having the same slump or flow. Fig. 12 may now be supplemented by Mr. Smith's Fig. 3A.

Mr. Smith has shown in an entirely satisfactory manner that the penetration and the remolding tests both bear the same relation to the flow test as long as the aggregate gradation is constant. However, it cannot be assumed that the penetration test would have the same reaction as the remolding test to changes in gradation, simply because it has the same reaction to changes in quantity and composition of the paste. We find that with gradation constant the difference between the remolding test and the slump test is also very small (although the agreement is not as good as that shown by Mr. Smith between the remolding and the penetration tests), whereas the response of the slump

test to changes in gradation does not agree with the remolding test at all.

The method of testing workability by measuring the power consumption of a mixer, proposed by Purrington and Loring in 1928[1] has many desirable features and its use was seriously considered for this investigation. However, certain mechanical difficulties, and what seemed to be some inherent weaknesses in the method caused us to adopt the remolding principle instead. One of the chief objections to the Purrington and Loring method is that it measures the resistance to shear only while the mixture is in a state of considerable agitation. It is felt that one of the requirements of a device for measuring workability is that it should test the mixture while the mix is in a state of agitation somewhat comparable to that found under average conditions of placing. This average condition is somewhere between the state of violent agitation found in the mixer and complete quiescence —probably nearer the latter extreme. It is well known that the leaner mixes exhibit far greater workability during or immediately after agitation than they do after remaining quiescent for even a very few moments. It was felt, therefore that the energy required for mixing, especially for lean mixes, may not be the same as for placing.

Messrs. Purrington and Loring state that one of the two factors which must be measured is the rate of application of the force. However, the tests on the three concretes to which they refer, indicate that when working with a given set of materials there is little to be gained from varying the rate of applying force. This will be seen from the curves of their Fig. 3[2] which are practically parallel for all three mixtures. In other words, the same conclusions regarding the relative workability of these three mixtures would have been drawn if the test had been made at, say, 24 revolutions per minute, or at some other number.

As to the actual workabilities of these 3 mixes, attention is called to the fact that these discussors have compared the mixes on the basis of constant flow, whereas, the writer drew his major conclusions from the relationship between remolding effort and factors of mixture composition. The discussors would, undoubtedly, agree that to make the 4000-lb. concrete appear as workable in their test as the 3000-lb. concrete, it would only be necessary to increase the flow of the 4000-lb. concrete by a small increase in paste content. They probably would agree also that having adjusted the consistencies of the two mixtures so as to offer the same resistance to shearing force, either could be handled

[1]Proc. A. S. T. M., Vol. 28, Part II, p. 499, 1928.
[2]Proc. A. S T M., Vol. 30, Part II, p. 659.

with equal ease on the job, so far as placing effort alone is concerned. The writer submits, however, that like the remolding test, the mixer test does not offer a complete measure of workability because it does not reveal the fact that under the conditions just described, the richer, 4000-lb. concrete would, undoubtedly, have superior workability by virtue of its greater cohesiveness and higher plastic limit.

Thus, it is seen that the mixer test has shortcomings fully as great, if not greater, than those of the remolding test. It keeps the mixture in a state of agitation hardly comparable to average conditions of placing, and it does not take into account differences in cohesiveness of mixtures which may offer the same resistance to shearing forces.

The illustration given by Messrs. Purrington and Loring, intending to show "the forces at work when workability and mobility are considered" does not seem applicable to the majority of concrete mixtures, nor does there appear to be a good reason for avoiding "dynamic forces." In the language of the writer's paper, the mixture of fine sand and water described would be one having an almost zero plastic limit. In other words, it would not be a truly plastic mixture except when in a state of agitation. Agitation causes temporary suspension of particles in such mixtures and thus aids workability. No experienced workman would attempt to move a concrete mixture of such characteristics by a steady push or pull; that is, by what the discussors have called "static force." Instead, he would, with his foot or his shovel, move the mass by a series of reciprocating motions which would put the mass in a state of mild agitation and cause it to flow more or less spontaneously. Why should this means of aiding mobility be ignored in the laboratory when it is so constantly used in the field? Why should the workability of a mass be defined as "resistance to static deformation," as the discussors have done? The forces applied in practice are not necessarily "static." Tapping the form, mechanical vibration, puddling by reciprocating motion, use of the strike bar, are all processes which impart vibration or "dynamic force" to the concrete.

It would perhaps be very desirable to have a device or devices which would measure the various factors contributing to workability in terms of fundamental units. For example, the degree of mobility would be measured in terms of force and flow; degree of plasticity, in terms of maximum unit plastic deformation; cohesiveness, in terms, perhaps, of cohesive forces and the mass and velocity of the particles tending to separate from the mix. But the well-nigh insuperable difficulties presented in such an approach to the problem are attested by the small degree of success with which efforts along that line have been rewarded. The problem is made doubly difficult because some

of the properties of a mixture, vital to workability, are transient and depend very largely upon the conditions under which the test is made, or under which the concrete is being placed. For example, it was brought out above that the amount of agitation imparted to a mixture affects its mobility. In the paper it was brought out also that a change in placing conditions can change the mobility of a mix.

Consideration of the vast amount of study which would be required to overcome all these difficulties, if indeed they could be overcome at all, has led us to accept for the present, a simple, relative test which embodies conditions of flow, degree of agitation, and acting forces somewhat comparable to average conditions found in the field. Just how well the remolding test meets these various requirements may be judged by the reader in the light of his own experience. (We feel that it needs to be supplemented by a test for cohesiveness). However, until means of measuring in absolute units are developed, the final criterion for judging must be common sense aided by observations as to the reaction of a device towards certain variables whose effect is known as to direction if not as to magnitude.

DISCUSSION OF THEORIES, EXPERIMENTAL RESULTS AND CONCLUSIONS

Discussion is made somewhat difficult because the different writers do not use some terms in the same sense. Messrs. Purrington and Loring, for example, have not used the same definition of workability as did the writer.

There are a number of views as to the proper definition of workability. According to one view, workability would be defined in terms of effort required to place a mixture; and according to another, workability denotes that quality of a mixture which permits its being handled without segregation.

As Messrs. Purrington and Loring pointed out, workmen, whose duty it is to handle the concrete, can but be of the opinion that workability is properly defined in terms of effort. On the other hand, the engineer, whose chief interest is quality, naturally gravitates toward the other definition. It will be seen that the definition used by the author embraces the belief that the term workability properly includes both the effort required to handle a mixture and its resistance to segregation.

The writer was surprised that Messrs. Purrington and Loring found it necessary to call attention again to the fact that the remolding test is not a complete measure of workability, inasmuch as this point was stressed in the paper. On page 423, for example, and on page 433, and yet again on page 445, there are statements which show that the author is in perfect agreement with the contention of the discussors.

It seems permissible, however, to state that a study of workability has been made, even though the study was confined to one phase of workability.

Like Messrs. Purrington and Loring, Professor Williams apparently did not take seriously the writer's repeated statement that the remolding test is not to be considered as a measure of workability, but as one of mobility under a given condition. After quoting one of the author's statements to this effect, Professor Williams, in the second sentence below the quotation, says: "These curves (referring to Fig. 11) show that a 1:3:6 concrete with a 2⅞-in. clearance is as *workable* as a 1:2:4 concrete with a 2⅝-in. clearance. . . ." etc. The author does *not* interpret Fig. 11 in this manner as will be seen by referring to page 445 of the paper. It is only the mobility under these test conditions that is the same.

As Professor Williams says, the tendency to segregate, which depends upon plasticity and cohesiveness, is an inherent property of a mixture. However, these curves support the statement on page 424 that mobility is not an inherent property of the mixture but is affected by the conditions under which the flow must take place. It should be clearly understood by this time that the number of jigs required in the remolding test is not taken as a workability figure but as an index to the mobility of the mixture under *the condition of the test.*

Early in this study it was recognized that there should be a test for segregation in addition to one for mobility. On the basis of field observations of the behavior of concrete being placed through improperly handled "elephant trunks" an apparatus was designed to simulate these conditions. It involved a down spout terminating in a joint held at an angle with the rest of the spout. The concrete was first placed in a hopper and allowed to fall downward through the spout. At the final stage of the fall it was deflected onto a plane surface, striking the surface at a small angle with considerable velocity. The coarser aggregate in the non-cohesive mixtures tended to separate from the mortar and travel the greatest distance from the end of the spout.

This method gave very good results with a variety of mixes but showed weaknesses. For example, a lean, fluid mixture with sufficient sand, would show surprisingly little segregation under the conditions of this test. Yet it was obvious that the mixture was not very cohesive and would probably segregate freely under many job conditions. It is believed that the method itself could be modified to give very satisfactory results but it is a question whether the results would be worth the effort, particularly in view of the possibilities of interpretation in

the remolding test, outlined in the third paragraph on page 445 of the February (1932) JOURNAL.

It cannot be conceded that Mr. Smith's Fig. 4 demonstrates that Celite itself increased the workability of the 1:1½:3 mix in question. By the statement that Celite caused a very noticeable increase in workability, Mr. Smith, no doubt, actually meant that Celite plus enough water to maintain constant flow caused an increase in workability. At no place in the paper has the author implied that such an increase in the paste content would not improve workability (see second paragraph from bottom, p. 433). It is merely stated that the indications were that the same changes in past volume and paste consistency brought about by the use of water and an appropriate amount of some other powdered material would produce practically the same results.

In this connection Mr. Smith's discussion of the author's Fig. 1 and Fig. 5 may be considered. Mr. Smith asks how "a single observation of flow rate at a single pressure may be expected to give an indication as to consistency." If Mr. Smith has in mind the rather complicated definition of consistency which involves the coefficient of mobility and the yield point, the method used did not give a true measure of consistency. It did, however, give an approximate relative value. Because of the imperfect nature of test, the author did not expect Fig. 5 to agree exactly with Fig. 1 as to the percentages of Celite and cement required to produce comparable pastes. Since the differences in results obtained from Celite and cement pastes may be largely erased by slight changes in proportions, the special properties of the Celite may still be considered to be a secondary factor determining the mobility of a concrete mixture.

The same statement may be made regarding Mr. Smith's comparison of Incor and regular cement. While it is not denied that there are differences inherent in these two types of cement, the data presented by Mr. Smith do not prove that by properly adjusting paste volumes and paste consistencies, differences in workability due to the kind of cement may not be reduced to minor importance. That is, a mix containing a finely ground cement has workability comparable with some richer mix of cement ground to ordinary fineness. To support the idea of the importance of an admixture's special properties, Mr. Smith cites the marked stiffening affect of adding lime to a Celite paste. He attributes this effect to formation of a gel on the surface of the particles. The direct result of this effect is that smaller amounts of Celite are required to produce pastes of given consistencies. Pastes of similar characteristics can be produced in distilled water by using higher

proportions of Celite. But in concrete the smaller proportions are required because the mixing water is quickly saturated with $Ca(OH)_2$ from the cement.

The stiffening effect of the lime is not necessarily due to a special property of Celite, nor is it at all certain that it is the result of the formation of a gel coating. Bentonite, and to a lesser degree, California Pumicite showed the same behavior as Celite. In any case a small quantity of acid will reduce a paste which has been stiffened by addition of a few grams of lime, to even greater fluidity than before adding lime; and then, the stiffer consistency can be restored by adding NH_4OH. This behavior is in line with the known flocculating or deflocculating effects of electrolytes in suspensions of finely divided solids.

Mr. Bates, in connection with this same section, points out what he considers uncertainty on the part of the author. There are several points on which the author is uncertain, for the study is by no means completed. However, in this particular case the uncertainty has to do with the secondary effects due to differences in cohesiveness of mixtures which have the same mobility (remolding effort) but which have pastes made with different materials. This is the phase of the problem on which we are now working.

The difference between Fig. 4C and that of Mr. Smith's Fig. 7, pointed out by Mr. Smith, is easily explained on the basis of differences in severity of placing conditions. The author used, in those particular tests, a ring clearance of 2⅝-in. and a maximum size aggregate of 1½ in. Mr. Smith used a ring clearance of 2¾ and ¾-in. maximum size aggregate, a placing condition much less severe than used by the author. The results are in line with the discussion given on page 445 under the heading "The Use of Different Ring Clearance" and with Fig. 11 which is discussed on page 442. Use af a smaller clearance or larger aggregate in Mr. Smith's tests would have duplicated the author's results.

In view of the manner in which the author has handled the data it is difficult to see why Mr. Smith objects to the arbitrary consideration of the cement paste as a unit. All of the differences in the effect of changing either cement or water as brought out by Mr. Smith have been considered on pages 430 and 431 and illustrated in Fig. 4. It is true that when factors of mix composition are used as a basis of comparison, rather than the flow, there is left only the remolding effort to give an indication as to the working qualities of the mix. This, however, should not leave one completely in the dark in this respect. On the other hand, when the flow is used as a basis of comparison it is

very easy to lose sight of the fact, just as Mr. Smith has done, that the various mixtures of given mobility may not be comparable as to their suitability for a given work. A consideration of workability is of small practical value unless the comparisons are made between mixtures that are otherwise suitable for a given job.

Mr. Bates questions the reality of suspension in workable concrete mixtures. A theory was presented in the paper intended as an explanation of this phenomenon, but it apparently did not meet with Mr. Bates' approval. The author is perfectly willing to discard that explanation whenever its fallacy is proved or whenever a more acceptable one is offered. Any theory offered must explain the following phenomena:

(1) *A cross-section of hardened concrete which was workable before hardening shows that, in general, the aggregate particles, even the sand particles, are separated by layers of paste at the time of hardening.* This being the case the aggregates cannot be considered to constitute a rigid inter-connecting framework in which the paste (or water) is suspended, as Mr. Bates has implied. Since the heavy particles are not in contact they must be suspended by the lighter paste, and if the heavy aggregate particles are so suspended by the paste, is it not equally likely that the much finer cement particles bear a similar relation to the water?

(2) *When a particle of aggregate is dislodged from hardened concrete its matrix will be found to be composed entirely of hardened paste, no aggregate being visible.* This is further evidence that the aggregate particles do not form a connecting framework in the concrete.

(3) *Fresh concrete exerts an amount of lateral pressure against the forms for a considerable period after placing, the same as if it were a liquid of the same specific gravity as the concrete mixture.*

(4) *After being placed. the solids in workable mixtures settle uniformly and displace water upwards.* Since there can be no appreciable change laterally in the positions of the particles to a closer-packing arrangement, the particles must not have been originally in contact with one another, or there could be no uniform settlement.

Thus, to resort to an old device, if the solids are not suspended in the water they act exactly as if they were.

Perhaps Mr. Bates has found the author's explanation of suspension unsatisfactory on account of the use of the term surface tension. Continued study of the subject has shown that that term, which was intended to indicate the tension between the liquid and the solid, is,

by convention, applied only to surfaces in contact with gas. It appears that the term adhesion tension should have been used.[3]

Regarding Mr. Bates' general comments on the use of words, the author was careful to indicate just how the more important terms were being used in the paper. It is realized that some of the words take on special meanings when used in connection with certain sciences. In general we tried to use the meanings most likely to be in line with the vocabulary of the general reader of engineering literature.

The use of the word "now" on page 434 may be taken to refer to the author's own evolution of thought. The author does not see as close an agreement between his conclusion and that of Professor Abrams as does Mr. Bates. Granting, however, that there is room for differences in interpretation of Professor Abrams' paper it must be agreed that the results could not be considered conclusive because the actual workability of the mixtures was unknown. With the present paper the conclusion must still be tentative, pending the development of a test which will measure *all* phases of workability, or of a test supplementary to the remolding test which will take fully into account plasticity and cohesiveness.

In connection with this question, Mr. Bates has inaccurately quoted the paper by including only a part of the sentence in his quotation. The last phrase, which Mr. Bates omitted, is important for its refers to the conditions of high water-cement ratio and low percentages of sand which are necessary to observe the effect of admixtures at their best advantage.

Mr. Bates might have mentioned also that Professor Abrams stated in Bulletin 1 of the Structural Materials Research Laboratory, Lewis Institute, that a wide variety of aggregate gradations could be used. Thus with only the slump test and observation of mixtures as a guide he reached practically the same conclusion as arrived at in this paper. Our tests are notable only in that they are based on the results of a mechanical test and that they systematically tested a wide variety of size combinations.

The last paragraph of Mr. Bates' discussion indicates that he might have become a little too optimistic over the possibilities of the remolding test as a field apparatus. Turning to the paragraph on page 441 to which Mr. Bates refers, it will be seen that the test was recommended for designing mixes only under very restricted conditions—conditions which are rarely found in the field.

[3]Herbert Freundlich, Colloid and Capillary Chemistry, page 157.

Mr. Bruce mentions the possible effects of such factors as temperature, time, chemical changes, and time of mixing. All of these may be interpreted as factors affecting the consistency of the paste and it is difficult to see how taking these factors into account could alter the author's conclusions regarding the effect of paste consistency.

The author agrees with Mr. Bruce that correlation of laboratory and field tests are desirable and that the studies of Kellerman and Jackson are most valuable. However, one of the functions of a laboratory is to obviate the necessity for expensive field tests. By combining experience, common sense, and laboratory results, the problems of workability can be made too small to warrant elaborate field tests.

"Reinforced Concrete Column Investigation"

*Discussion of Progress Report of Committee 105**

F. E. Richart, Chairman Committee 105†—The program of column tests conducted by Committee 105 is nearly completed. There remain a few columns only which are being studied under long-continued loading, and observations on these may be continued for another year or more. The results already derived from the tests of nearly 600 columns furnish not only a fund of new information but also verify general relations previously established on a more restricted range of materials. The present tests, with a wide variation in strengths of concrete, kind and amount of both vertical and spiral reinforcing, size and design of columns, and age and storage conditions, demonstrate that the ultimate column strength is governed in all cases by a simple general law. The strength has been divided into three elements, contributed by the concrete section, the vertical reinforcement, and the spiral reinforcement, according to rational and well-established laws of mechanics.

In two particulars, there is a difference in the results from the two laboratories. The spirally reinforced columns tested at Lehigh University had in general a lower strength than those tested at the University of Illinois. This was evidently due to an unintentional difference in testing technique. At Illinois the spherical block at the top of the column was wedged after the initial load to produce the condition of a flat-ended column; at Lehigh the block was left free, giving more favorable conditions for lateral deflection near the maximum load. This resulted in an effectiveness of spiral reinforcement at Lehigh (0.7 to 2.0 times that of the longitudinal steel) lower than the values found at Illinois (1.6 to 2.4).

The other major difference is seen in the increase in the stress in the vertical reinforcement of columns under sustained load for a year. Starting from initial stresses of 8,000 to 12.000 p. s. i., the stress in-

*See *Proceedings*, Vol. 28, "Third Progress Report of Committee 105," p. 157; "Third Progress Report on Column Tests at Lehigh University," by Willis A. Slater and Inge Lyse, p. 159; "Third Progress Report on Column Tests Made at the University of Illinois," by F. E Richart and G. C. Staehle, p. 167; "Fourth Progress Report on Column Tests Made at the University of Illinois," by F. E. Richart and G. C. Staehle, p. 279; "Fourth Progress Report on Column Tests at Lehigh University," by Inge Lyse and C. L. Kreidler, p. 317.

†The discussion by F. E. Richart and others, except as noted, was presented at the 28th Annual Convention, Washington, D. C., March 1-4, 1932.

creased due to time yield of the concrete to extreme values of 30,000 p. s. i. at Illinois and over 40,000 p. s. i. at Lehigh. This difference has been attributed to the use of different aggregates and to different atmospheric conditions at the two places.

The sustained load tests employed working loads computed by use of the present A. C. I. formula, which permits relatively high loads on columns having a high percentage of vertical steel. This evidently accounts for the fact that the steel stress in all columns tends to approach a common value after a year's time, as intended by the originator of the formula. In none of the columns under sustained loading was the yield point of the vertical steel reached. This is reassuring in that it indicates that with steel of the intermediate grade used, or of higher strength, there should not be undue deformation of the column, such as might be anticipated if the yield point were passed and both concrete and steel reached a stage of plastic yielding.

In a later report the committee proposes to sum up and give its interpretation of the principal results of the investigation, and to make recommendations, based on its findings, as to design formulas for columns.

*H. J. Gilkey**—It seems to me one point has not been made quite clear. If the spiral is twice as effective, why not omit the longitudinal steel? Of course we need the longitudinal steel; perhaps Professor Richart will discuss that.

F. E. Richart—I will try to state briefly the functions of vertical and spiral reinforcement. The vertical reinforcement takes direct compressive stress due to the applied loads; it adds greatly to the resistance of the column in bending, and as shown by our tests of Series 3, it offers resistance to the undesirable shortening of the column due to time yield and shrinkage of the concrete. The spiral reinforcement offers little or no resistance to bending or to the shortening due to time effects; it lies dormant in the column until the yield point of the column as a whole is reached, when the concrete bulges sufficiently to bring the spiral into play. The strength added to the column by the spiral is thus accompanied by large inelastic deformations, and spalling of the shell. This strength is a reliable and definite part of the column strength, and because of the large deformations required, failure comes slowly and with ample warning, instead of the sudden failure to be found in plain and tied columns. Obviously the vertical steel is essential to carry stress and to minimize distortion of the column at working loads, while the spiral steel is very effective in building up the ultimate strength and in preventing a sudden failure.

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*M. A. Gerwig** (by letter).—In 1924 the Joint Committee gave us a working tool with which to design reinforced concrete spiral columns—

$$P = A_c\,[1 + (n - 1)\,p]\,[300 + (0.10 + 4p)\,f'_c]$$

This formula was evolved by our leading engineers from the best data at the time available. It has been adopted by the American Concrete Institute and has been incorporated in many of our large city building codes where it will be in force many years from today.

Any formula should have two characteristics: (1) It should express the foundation on which it rests, be that foundation theoretical on empirical; (2) it should be usable, that is what it is made for, to be used. Whether this formula possesses the first quality or not the writer does not feel qualified to state but what does the world at large think about the second quality? Eight years after the adoption of this formula, in February 1932, there appeared in one of the current engineering magazines an attempt to produce a usable solution. All published attempts have thus far fallen short, they involve "cut and try" methods, interpolation or laborious and treacherous backtracking to determine the quantities required for a given load. The formula is unwieldy, cumbersome, awkward, irksome and altogether unmanage-

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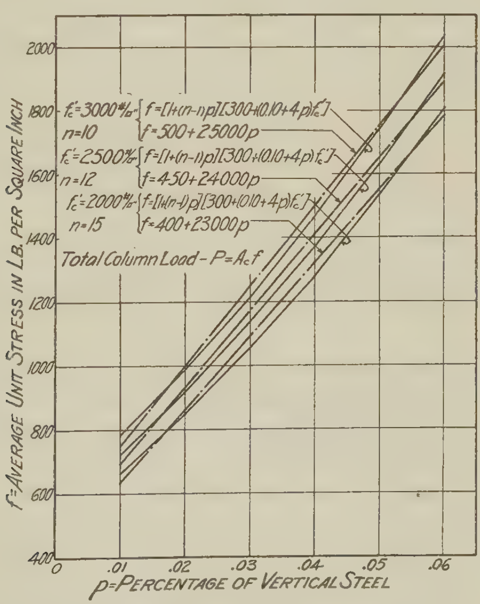

COMPARISON OF SIMPLIFICATION FOR SPIRAL COLUMN FORMULA,
$P = A_c\,[1 + (n - 1)\,p]\,[300 + (0.10 + 4p)\,f'_c]$
STANDARD WITH JOINT COMMITTEE, A.C.I. AND MANY PRINCIPAL CITIES

able. For years it has defied reasonable tabulation or graphical presentation.

Yet this need not have been so. If the individual members of the committee who instigated the formula had had personally to design a dozen multiple story buildings using it, it is safe to state that it would *not* have been so.

The accompanying graph shows that a straight line formula could have been used giving values with an average variation of about 2 per cent from those obtained by using the complicated formula. A straight line formula admits of easy, direct, usable charting or tabulation. If the values obtained from the formula are acceptable, certainly those gotten from the straight line would be satisfactory also.

From the results of the Lehigh-Illinois tests it is anticipated that shortly a new column formula will be derived. It should be comprehensive, certainly. And it should be usable. Let us have a tool designed for the job.

Readers are referred to this Journal *for January 1933, for a continuation of this discussion which should reach the Secretary by Nov. 1, 1932.*

Discussion of Report of Committee 312:

"Plain and Reinforced Concrete Arches"

This discussion of a report published in the Journal for March 1932, and presented at the 28th Annual Convention in Washington was scheduled for this Journal, but will be published next month—October issue.

Discussion of Report of Committee 902:

"Proposed Specifications for Concrete Pavement in Municipalities"

This discussion of a report published in the Journal for March 1932, and presented at the 28th Annual Convention in Washington was scheduled for this Journal, but will be published next month—October issue.

The Freyssinet Method of Arch Construction Applied to the Rogue River Bridge in Oregon*

BY ALBIN L. GEMENY† AND C. B. MCCULLOUGH‡

The fixed concrete arch is one of our most desirable types of bridge structures because of its susceptibility to artistic treatment. However, when constructed by the methods now commonly used in this country, this type of structure possesses certain inherent defects which greatly narrow the range of span lengths, rise ratios and foundation conditions which can be safely and economically utilized. Deformations and the resulting stress changes are produced in the arch by volume changes in the concrete due to elastic and plastic shortening under load, shrinkage while drying, temperature changes, and movements of the supports. To provide for these deformation stresses in the design is costly in all but very short spans of high rise ratio and becomes impracticable in moderately long spans of low rise ratio. Furthermore, approximations in the computation of dead and live load stresses which are safely used for short spans may give rise to serious inaccuracies for long spans.

Failure to provide for deformation stresses is likely to result in unsightly cracking even in comparatively short spans. In long spans there is danger of failure from this cause. Obviously, any means which permits a positive control and adjustment of strains in the arch after its completion does much to eliminate uncertainties of design and construction and greatly increases the utility and economy of the arch bridge.

The French engineer, Freyssinet, conceived the idea of neutralizing to a great extent these deformation strains and reducing total strains by introducing into the arch compensating deformations and strains. In 1908 he built an experimental two-hinged concrete arch for an investigation of his idea. Since that date a number of arch bridges have been built in France by the method which he developed and span lengths have been gradually increased until the record span of 612 ft.

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*Presented at 28th Annual Convention, March 1-4, 1932, Washington.

†Senior Structural Engineer, U. S. Bureau of Public Roads.

‡Bridge Engineer, Oregon State Highway Commission,.

FIG. 1—FINISHED STRUCTURE LOOKING UPSTREAM

was reached in the recently completed structure at Brest, France.[1] Mr. Freyssinet thinks that concrete arches are competitive with other types of bridges of longest spans. Because of his past achievements, his opinions, however apparently fantastic, merit the serious attention of American bridge engineers.

The Freyssinet method had not been tried in this country until it was introduced by the Bureau of Public Roads and the State of Oregon in the construction of the Rogue River bridge on the Roosevelt Highway, Federal Route 124B, in Oregon to determine to what extent this method of strain adjustment can be employed to advantage and, incidentally, to learn the behavior of an arch when it is first put under dead load.

By the Freyssinet method, the compensating strains are produced with batteries of hydraulic jacks placed in one or more radial joints in the arch ribs. In unsymmetrical arches two joints generally should be used. For symmetrical arches a joint at the crown is sufficient unless considerable foundation movement is anticipated when provision should be made for counteracting it by rotating the arch at the springing line. In this manner, positive control is exercised over the axial separation and angular rotation of the joint faces, or over the magnitude and line of action of the normal thrust at the joint. Mr. Freyssinet calculates the joint opening which theoretically will produce the most favorable elastic state throughout the arch and then produces this opening with the jacks. Rib shortening, which occurs before jacking, may be compensated simply by opening the joint by an amount equal to the shortening. Anticipated future changes in rib length, due to shrinkage and to temperature changes from the keying temperature to the mean temperature assumed in the design, may be compensated by introducing strains which produce

[1] "The 600-ft. Concrete Arch Bridge at Brest, France," by E. Freyssinet, translated by S. C. Hollister, *Proceedings*, Am. Concrete Inst., Vol. 25, 1929, p. 83.

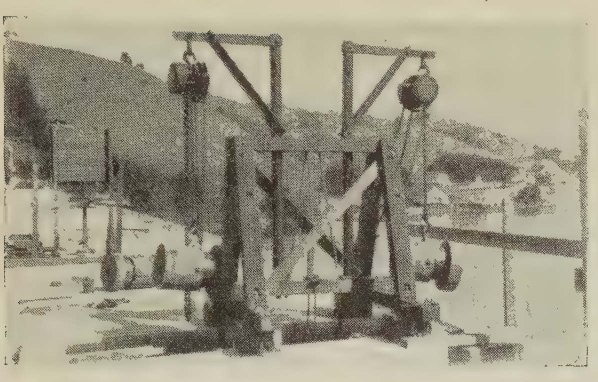


FIG. 2—LOWERING JACKS INTO PLACE

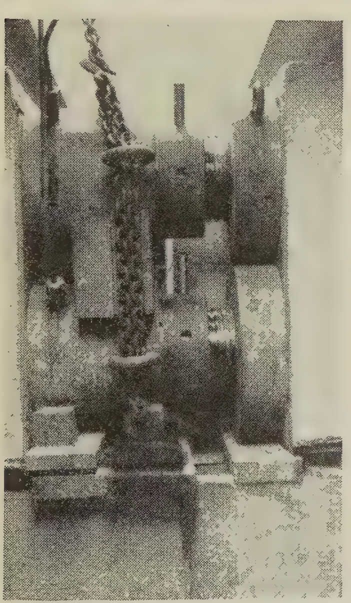
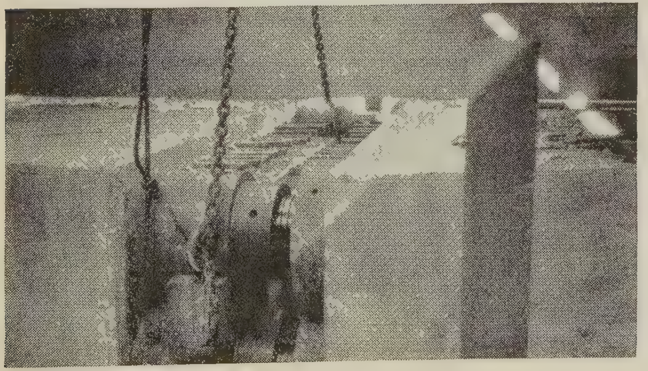


FIG. 3—JACKS (LOOKING AT THE SIDE OF RIB) SHOWING SAFETY RINGS BEARING ON THE BODY OF JACKS

FIG. 4—JACKS IN PLACE IN CROWN JOINT



stresses counter to those which will be produced by these anticipated changes in rib length.

So far as we know, there is no published record of deformation and strain measurements made to show how closely Mr. Freyssinet achieved the desired results; nor are there data on the progressive deformations which occur while the arch is being jacked from its neutral position on the centers to its final position. Apparently, he depended entirely upon the calculated relation between the joint opening and strains on the extrados and intrados at the critical points on the rib. This procedure is probably accurate for the ribs without superstructure. However, superstructure restraint will change the relation between joint opening and local rib strains. We believe it is entirely practicable to produce the most favorable elastic state in an arch with superstructure in place by determining in the field, by trial, the relation between strain gauge measurements at critical points on the rib and the joint openings, produced by the jacks, although it is simpler and generally more desirable, from a construction viewpoint, to make a preliminary adjustment on the ribs and a final adjustment on the completed structure.

The Rogue River bridge consists of seven 230-foot symmetrical 2-rib arch spans with a rise of 47 ft. Two anchor piers divide it into three independent groups of spans: A central group of three spans with two intermediate elastic piers and a two-span group at each end with one intermediate elastic pier. The abutments are on solid rock. All piers are on timber piles driven to a penetration of about 30 ft. in a sand-gravel soil. Battered piles are provided to take the horizontal component of arch thrust on the anchor piers.

The concrete in the ribs was made of a high grade local sand and gravel derived from basaltic rock. The coarse aggregate was separated into two sizes. The mixture was proportioned for a strength of 5000 p.s.i. in 28 days. This strength was obtained within narrow limits of variation. The proportions were one part cement and 2.75 parts aggregate, a water cement ratio of .593 to .653, cement content 8.48 sacks of cement per cu. yd. Rigid control of the proportions was maintained on the job, changes being made in the proportions of sand to gravel and in the water cement ratio when necessitated by a change in the character of materials. (See Table 1). The reinforcement consisted of 15-1¼-in. sq. bars of intermediate grade steel, at both top and bottom, from the piers to the quarter points and 15 1-in. sq. bars from the quarter points to the crown. The percentage of reinforcement was 0.71 at the skewback and 1.23 at the crown

TABLE 1—RESULTS OF TESTS ON RIB CONCRETE

1	2	3	4	5	6	7
Cyl. No.	W/C	P	S28 Calculated	S28 Actual	Residual	E
52	.593	.340	5130	5625	+495	
53	.613	.370	5340	5085	−255	4.68
54	.643	.373	5070	4970	−100	4.74
55	.604	.325	5870	5730	+860	4.85
57	.653	.384	5100	4845	−265	
58	.631	.365	5040	5480	+540	4.53
59	.599	.363	5400	5960	+560	4.87
60	.637	.358	5000	5375	+375	4.78
61	.629	.370	5190	4760	−430	4.83
62	.600	.335	5000	5220	+220	4.38
63	.622	.373	5300	4860	−440	4.62
64	.600	.386	5640	5000	−640	4.97
65	.616	.363	5250	4835	−415	4.87
66	.608	.392	5670	6190	+520	4.96
67	.620	.364	5220	6065	+845	4.67
68	.613	.351	5130	4985	−145	5.37
71	.609	.380	5490	5695	+205	4.96
72	.622	.360	5160	4800	−360	4.83
73	.592	.353	5270	5780	+510	4.99
74	.610	.357	5220	4795	−425	4.89
77	.604	.366	5360	5445	+ 85	4.77
78	.582	.342	5220	5170	− 50	4.98
79	.607	.357	5220	4985	− 65	4.55
80	.597	.349	5190	5170	− 20	4.40
81	.612	.365	5270	5895	+625	4.48
82	.629	.380	5340	5320	− 20	4.38
83	.585	.344	5220	4910	−310	4.31
95	.608	.365	5340	5055	−285	3.79
96	.604	.376	5480	5745	+265	3.94
98	.584	.371	5580	5270	−310	4.06
100	.590	.374	5580	4950	−630	4.19
102	.566	.359	5580	4935	−645	4.35

Col. 2—Water cement ratios from analyses made by Dunagan Method.

Col. 3—Proportion of paste (cement plus water) from analyses made by Dunagan Method.

Col. 4—Calculated strength from $S_{28} = \frac{30000\ p^{85}}{4.2\ W/C}$.

Col. 7—Specimens 95 to 102 tested at ages of 14 to 30 days; other specimens from 76 to 98 days old.

exclusive of the steel in the jacking bracket. The design stress for the rib concrete was 1200 p.s.i.

The pier concrete was designed for a strength of 2500 lb. at 28 days with a working stress of 650 p.s.i.; the superstructure concrete for a strength of 3000 lb. with a working stress of 800 lb.

In placing the rib concrete a 10 ft. section was omitted at each skewback until the superstructure was placed so as to prevent cantilever action due to settlement of the centers. These sections were placed two weeks before jacking started.

Temporary open joints were left at the crown of each span for jacking. The ribs were widened at the joint so as to provide enough cross section for the jack emplacements and a sufficient net section for keying between the intrados and extrados steel. The reinforcement was temporarily left unconnected across the joint. A structural steel jacking bracket extended about 10 ft. into the rib back from the joint.

The adjusting operations were executed simultaneously on two spans. A battery of eight jacks was placed in each span and connected to hand pumps so that the extrados and intrados jacks could be operated independently. Four accurate pressure gauges were provided for each battery of jacks,—two for the extrados jacks and two for the intrados jacks.

The exigencies of construction made it expedient to perform the adjusting operations after the superstructure was placed. In order to reduce, as far as possible, superstructure restraint of the ribs, temporary joints and bearings were placed at each panel point except the point nearest the skewback where the spandrel column, because of its length, was considered sufficiently flexible to avoid appreciable restraint at this point. In spite of these precautions, superstructure restraint greatly reduced the deformations as computed for free ribs.

Special jacking equipment had been developed in France in connection with the work of Mr. Freyssinet. In his early work steel plates were inserted in the joints as they were opened by the jacks to avoid the danger of a sudden release of pressure in the jacks. The use of these shims was not necessary with the jacks used on the Rogue River bridge as they were equipped with safety rings threaded on the piston. This ring can be screwed down to a bearing on the body of the jack and can safely carry the entire jack load when the pressure is released. By keeping this ring bearing against the body of the jack at all times as the piston moved, all danger from a sudden release of the pressure was removed. The arch thrust was carried on these rings whenever pumping was interrupted. The jacks were provided with a spherical bearing at the head to allow rotation of the bearing faces.

The jacks had a rated capacity of 275 tons each, or a total capacity of 4,400,000 lb. for each span. The computed dead load thrust of the arch was about 1,800,000 lb. There was sufficient jack capacity to permit the application of the entire dead load thrust at either the extrados or intrados alone. The pumps had a capacity of 10,000 p.s.i.

The jack thrusts were applied in small increments. For each increment of thrust careful measurements of movements and local strains were made as follows:

1. Jack thrusts on extrados and intrados of both ribs.
2. Spread of the crown joint on both ribs.
3. Vertical movements of the crown of the east rib and of the piers.
4. Rotations of the east rib at the crown on both sides of the joint, at the north quarter point, at the skewbacks and at the center of each pier.
5. Horizontal movements of the piers parallel to the center line of bridge, at the elevation of the springing line.

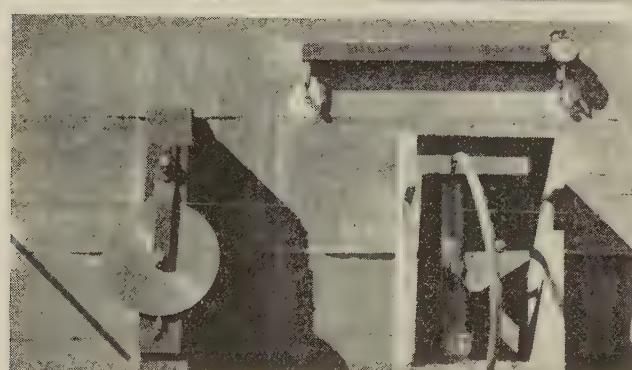


FIG. 5—PRESSURE GAUGES, DISTRIBUTOR BLOCK, PUMPS AND PIPING TO JACKS

FIG. 6—WATER LEVEL GAUGE TUBE, HORIZONTAL WIRE AND LEVEL BAR IN PLACE

6. Strains in the concrete on the extrados and intrados of the east rib at a point 10 ft. from the crown joint and back of the jacking bracket and also at both quarter points and at both skewbacks.

These measurements were made as follows:

1. Jack thrusts were determined by Bourdon pressure gauges accurate to 20 p.s.i. with a maximum capacity of 7500 p.s.i. A check gauge was provided for each independent group of jacks.

2. Crown spread was measured with a sliding scale and vernier reading to .01 in. at four points on each rib—two near the extrados and two near the intrados. Gauge points were placed on the body and head of each jack so that the piston movements were measured. The upper and lower points were in planes $25\frac{3}{16}$ in. apart vertically.

3. Vertical movements were measured from a water level established by means of a closed system consisting of a 2-in. water line connected with a $\frac{3}{8}$-in. air line, both supported on the falsework caps and extending 2000 ft. over the entire length

of bridge. Closed gauge tubes were connected to the water and air lines at points where vertical movements were measured. The closed system with air line prevented fluctuations of the water level because of barometric changes. Volume changes of the water were reduced to a very small amount by large shallow expansion pans placed at intervals in the water line. Weighted invar wires were suspended from the rib. Measurements were made between the water level in the gauge tubes and a point on the suspended wire by means of a graduated sleeve sliding on the tube. A base gauge tube was placed on an abutment at one end. Readings were taken on the base tube for each series so that correction could be made for changes of level due to leakage or accidental injury to the pipe line.

4. Rotations were measured with level bars. Crown rotations were measured on top of the rib at the center. Rotations at other points were measured on the side of the ribs at the axis.

5. Horizontal pier movements were measured from wires supported on pulleys in the side of the piers at the elevation of the springing line. A wire was anchored at each shore and counterweighted at the center piers. Measurements were made with a scale between points fixed on the wire and in the sides of the piers. Nickel-steel wire with a small coefficient of expansion was used.

6. Strains in the concrete were measured with the cartridge type of McCullom-Peters electric telemeters embedded in the concrete just outside the reinforcement on the extrados and intrados. Temperature coils were placed on the axis of the arch at each section where the strains were measured. All telemeters were wired into a central switch board. Telephone communication was provided between the deck of the bridge and the switch board which was located under the bridge on the falsework.

The coefficient of expansion of the concrete was determined on a cylindrical specimen sealed in a tin container to prevent change of moisture content. Volume changes in the specimen were measured with a telemeter embedded at the center. The specimen was placed in a water bath and run through a range of temperature of about 80°C. The coefficient was found to be .0000096 per degree Centigrade or.0000053 per degree Fahrenheit. This value was used in all computations in which temperature was a factor.

To determine the maximum shrinkage of the concrete which might be expected, a 6 by 12 in. cylinder was cast with a telemeter at the center. This cylinder was protected from direct wetting from rain but was otherwise exposed to the same atmospheric conditions as the arch ribs. Periodic observations of volume changes were made over a period of several months. It was noted that, shortly after placing, the temperature of the specimen began to rise and the telemeter showed expansion. After the heat of setting was dissipated a decrease in volume began and continued over the entire period of observation. At the end of three months it had reached a value of about .0007 in. per inch.

Readings on the telemeters and temperature coils in the ribs were begun immediately after placing of the concrete and continued until

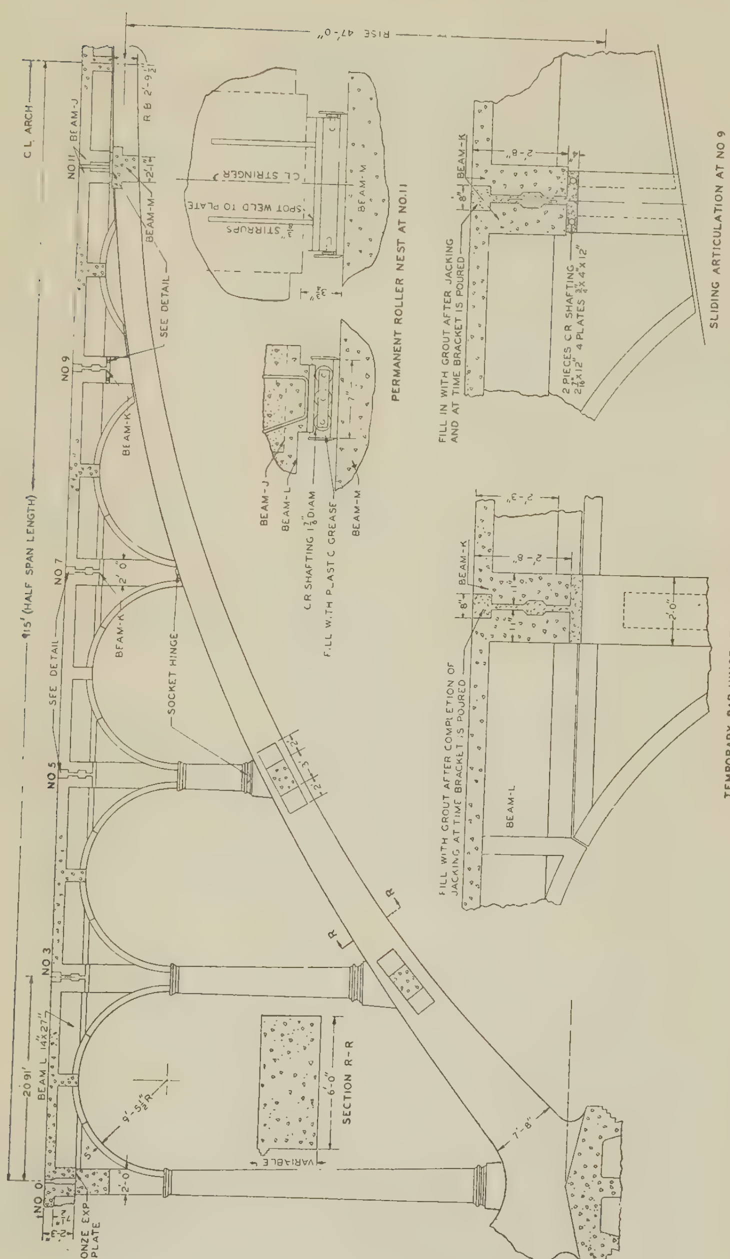


Fig. 7—Half section of arch with details of superstructure articulation

the adjustment operations were started. A full size unrestrained rib section, 6 ft. long, with full percentage of reinforcing steel, was placed alongside the bridge for comparison with the ribs. Telemeters were placed in this section as in the ribs. It was observed that a shrinkage of about .00015 in. per in. had occurred in the ribs at the time of jacking, .00055 less than the maximum possible shrinkage of the plain concrete. Practically the same results were obtained from the ribs and the free, full size specimen of rib.

Temperatures at the axis of the rib increased as the concrete set up and reached a maximum about 30 hours after placing. The maximum temperatures varied from 120° to 140° F. with air temperature at about 60°. The temperature of the concrete reached air temperature in about 100 hours after placing.

Considerably more study must be given the data which were gathered before complete conclusions can be drawn. A detailed report of the entire job will be given later in *Public Roads*[2]. In this paper we present the results of observations on span 7, with conclusions, which are subject to modification when the data for all spans are studied and correlated.

The sets of readings are numbered consecutively for the entire bridge. These numbers are shown on the diagrams.

In Fig. 9 the jack thrusts are plotted against the crown spread. It may be noted that, as the jack thrust increased, the crown opening slowly increased until position 101 was reached, where for the first time there was perceptible separation of ribs and forms. Obviously, the centering and falsework were undergoing elastic recovery because at 101 the crown had risen almost 0.5 in. while the crown had opened about 0.65 in. The suddenly increased crown opening between 101 and 102 is not reflected in the deflection shown in Fig. 10 and is apparently due to the destruction of the frictional resistance of the forms to rib shortening when the ribs were lifted clear of the centers. Fig. 11 shows the deflection-crown opening curve practically flat between 101 and 102.

As further jack thrusts are applied, both the crown opening and crown deflection curves flatten rapidly until at 109, where the thrust is 1,040,000 lb., they are practically flat. In this position, very small jack thrusts are required to lift the arch and increase the crown opening. Fig. 11 shows that between 101 and 106 the crown opening was more rapid than the rise. From 106 to 108 the rise was the more rapid. Between 108 and 109 the two increased at about the same rate. It will be seen later that significant changes in stresses and rotations occurred at position 106. The thrust in this position was 960,000 lb. The theoretical dead load thrust was 856,000 lb. in position 101 where separation of ribs and forms was first observed. Practically no rotation of the joint faces occurred until position 106 was reached when rotation began at a rapid rate. No such changes in rotations were observed at the quarter point and skewbacks where the rotations increased from the beginning of jacking to position 109 without sudden changes.

[2]Bureau of Public Roads, Washington, D. C.

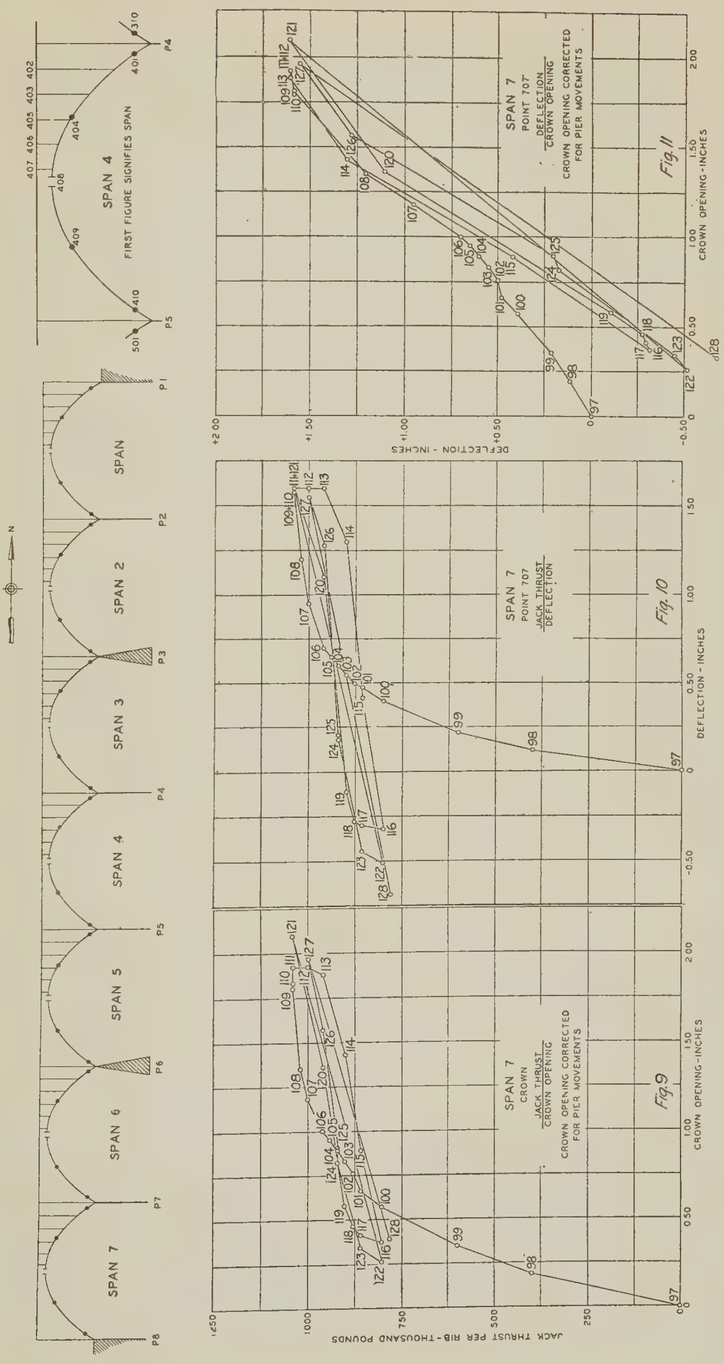


FIG. 8 (TOP), 9, 10, 11

At 109 the arch was left on the safety rings for about 20 hours while the wedges were removed and the arches swung entirely free. Until this was done, there was a short length of rib near the skewbacks still in contact with the centers because of the slight rise of this part of the arch. The effect of this support at the skewbacks was not great because no measurable change occurred in elevation of crown when the wedges were removed.

At 111 the jack thrust was brought back to 1,040,000 lb. Fig. 9 and 10 show a crown spread of about 0.12 in. with a very slight change in elevation. Apparently, this increased crown opening represents the permanent or plastic shortening of the rib during the period of 20 hours on the rings. This checks very closely the plastic shortening which was determined for spans 1 and 2 by lifting the arch to the highest point and after allowing it to remain on the rings for about the same length of time, placing it again on the centers in an unstressed condition.

Between 111 and 113, the jack thrust was reduced 100,000 lb. with a very slight decrease in the crown opening and no vertical drop at the crown. This lag in the movement of the arch may be explained by the friction of the articulations and possibly, to some extent, by jack friction although calibration of the jacks disclosed no appreciable friction. As the pressure was further reduced, the crown opening and deflection changed in proportion to reduction in jack thrust.

Between 113 and 116, a diminution of jack pressure of 160,000 lb., resulted in a proportionate closing of the crown joint of 1.50 in. and drop of the crown of 1.95 in. At 116 the crown was 0.35 in. below its original position on the centers.

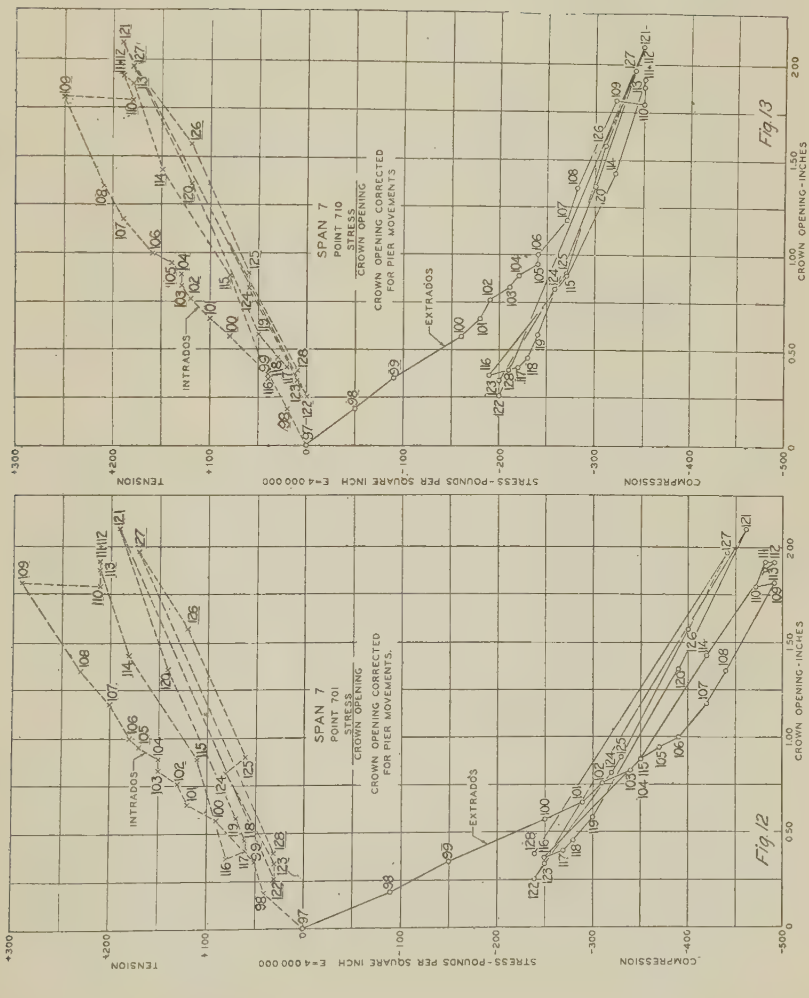
The jack thrust was now increased. Again the crown opening and deflection lagged between 116 and 117, very slight movement being produced by an increase in thrust of 75,000 lb. From 117 to 121, a range of jack thrust of 160,000 lb., there was a uniform spread and rise of 1.80 in. and 1.75 in., respectively. The jack pressure was once more released. No intermediate readings were taken between 121 and 122 and, therefore, the lag at 121 was not detected. Again the lag is seen between 122 and 123 on the upward run. From 123 to 127 the movement is uniform.

At 128 the adjusting moment and thrust was produced by the jacks which resulted in a lowering of the crown to a point 0.68 in. below the original position on the centers.

In Fig. 11, at zero deflection, when the crown was at its original elevation on the centers, it is observed that the maximum crown spread is 0.70 in. if we exclude the last run between 127 and 128, which is probably not accurately represented by the straight line because of the adjusting moment which was introduced between 127 and 128. This shortening of 0.70 in. included elastic shortening, plastic shortening and shrinkage which had been restrained by the reinforcement and centers.

In Fig. 9 we have noted that an apparent plastic shortening of 0.12 in. occurred between 109 and 111. This checks closely with results for spans 1 and 2 where the ribs were brought back on the centers in the neutral position. Furthermore, it seems logical to conclude that the difference, at zero deflection, of crown spread for the first run down after decentering and the last run up, as shown in Fig. 10 represents additional permanent shortening. The amount of this is about 0.10 in., making a total of 0.22 in. plastic shortening. Between 101 and 106, after the full dead load was on the arch, practically no rotation of the joint faces occurred. Taking the strain in the rib as the average of the telemeter readings at skewback, quarter points and crown, we find an elastic shortening of about 0.10 in. Of the total shortening of 0.70 in., we now account for 0.32 in., leaving 0.38 in. due to some other cause.

FIG. 12, 13


SPAN 7
POINT 710
STRESS
CROWN OPENING
CROWN OPENING CORRECTED FOR PIER MOVEMENTS
INTRADOS
EXTRADOS
Fig. 13
SPAN 7
POINT 701
STRESS
CROWN OPENING
CROWN OPENING CORRECTED FOR PIER MOVEMENTS.
INTRADOS
EXTRADOS
Fig. 12
TENSION
STRESS-POUNDS PER SQUARE INCH E=4 000 000
COMPRESSION
CROWN OPENING-INCHES

As shrinkage occurred in the rib concrete while still supported on the centers it was restrained by both steel and forms. We have seen that the preliminary telemeter readings indicated a shrinkage of about .00015 in. per in. which took place against the resistance of reinforcement and forms. Further shrinkage was prevented until the load was applied and the steel compressed an additional amount. Apparently this delayed shrinkage is the 0.38 in. by which elastic and plastic shortening is exceeded by the total shortening. This may be termed the elastic shrinkage shortening because in spans 1 and 2 it was shown that the resiliency of the compressed steel was sufficient to bring the rib back to its original length on the centers less the permanent shortening. Temperature changes during these operations were negligible. The total amount of shrinkage shortening, comprising that which occurred before and after jacking, was about .00028 in. per in.

The telemeters were very sensitive to the slightest jack pressure. After the slack in the jack pistons was taken up, the telemeters showed strain before any appreciable movement of the gauge indicator occurred.

Fig. 12 and 13 show stresses at the north and south skewbacks for each increment of load plotted against crown opening. These stresses are based on a modulus of 4,000,000 which was determined from laboratory specimens. It will be noted that from the beginning of application of jack pressures an apparent tension shows on the intrados and a compression on the extrados. The tension is roughly proportionate to the crown spread to 109, the position of highest thrust, while the compression shows a decelerating increase beyond 101, the position at which perceptible decentering was observed. The computed dead load stresses at the skewbacks are compressive on both intrados and extrados. This discrepancy can be explained by the existence of superstructure restraint. That such restraint exists to a marked degree, is indicated by the fact that computed elastic crown spreads and deflections are very much higher than the observed. As we shall see later, the discrepancy becomes less as we approach the crown and fewer spandrel column connections intervene between the jacks and telemeters.

At position 109, point 701, where the arches were left on the rings, the tension is relieved by about 75 p.s.i. and the compression by about 25 p.s.i. At point 710 the same decrease in tension occurs but an increase occurs in the compression. At point 701, the maximum tension after position 110, where all support of the centers is removed, is about the same as for point 710 but the compression is about 125 p.s.i. greater. However, this difference in the north and south halves of the rib disappears in the adjusted position at 128. The range of tension produced on the intrados between the extreme portions is about 175 p.s.i., while on the extrados there is a range of 225 p.s.i. compression. Within this range, the relation between crown spread and stress is a straight line relation to within 25 p.s.i. In the final position at 128 there is a tension of 25 p.s.i. at 701 and 10 p.s.i. at 710 on the intrados. On the extrados, there is a compression of 240 p.s.i. at 701 and 210 p.s.i. at 710.

At 701 the calculated dead load stresses are 227 p.s.i. compression at the intrados and 147 p.s.i. compression at the extrados. For a thrust of 856,000 lb., at position 115, the theoretical dead load thrust, the corresponding measured stresses are at the intrados 110 p.s.i. tension and at the extrados 350 p.s.i. compression. Theoretically, the dead load thrust line was 3 in. below the arch axis. At position 115 the thrust line was actually 29 in. above the axis. In the final position 128, the thrust line was lowered to a point 15 in. above the axis. Further shrinkage or temperatures below the mean will still further lower the thrust line and eliminate tension on the intrados.

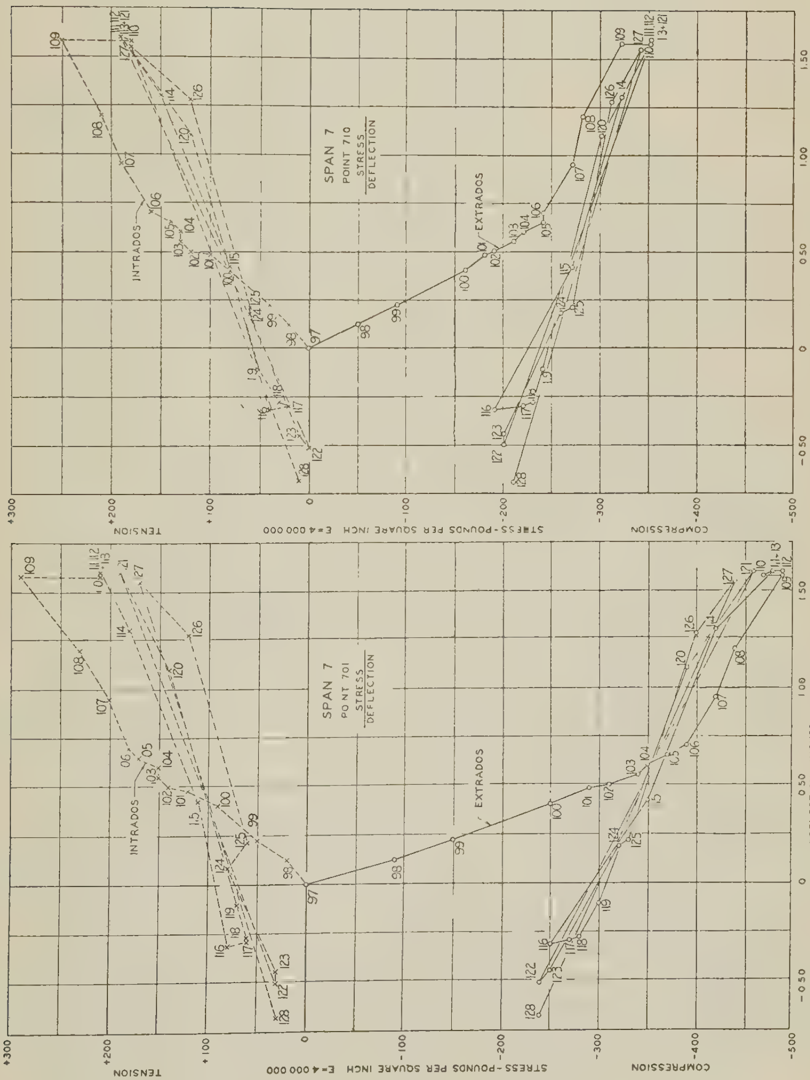

SPAN 7
POINT 710
STRESS
DEFLECTION
SPAN 7
POINT 701
STRESS
DEFLECTION
INTRADOS
EXTRADOS
TENSION
COMPRESSION
STRESS-POUNDS PER SQUARE INCH E=4 000 000
DEFLECTION - INCHES

Fig. 14, 15

FIG. 16, 17, 18, 19

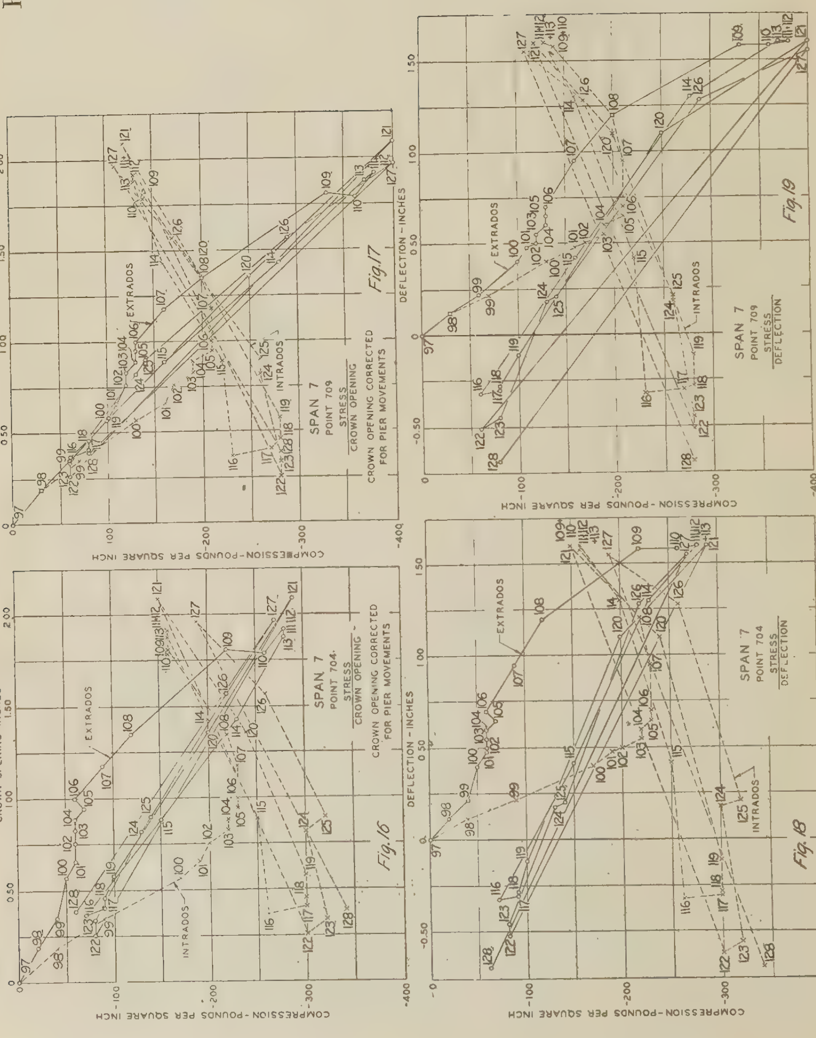


Fig. 14 and 15 show extrados and intrados stresses for point 701 plotted against crown deflections. We have here the same relation as between the stress and the crown opening. These diagrams show even a closer proportionality between stress and deformation.

Fig. 16 and 17 show compression on both extrados and intrados at the quarter points. The intrados compression at 704 increases rapidly to 225 p.s.i. at position 105 where, after a further crown spread of 0.25 in. the compression decreases to 160 p.s.i. at 109. Compression increases slowly on the extrados to 106 when it begins to increase more rapidly while the intrados compression is decreasing. The extrados compression at 709 is about 125 p.s.i. greater than that at 704 but, as in the case of the skewback stresses, they become equal in the adjusted position at 128. The variation from proportionality of stress and deformation is again within reasonable limits although it is somewhat greater for 704 than for 709. The final stresses here are: Extrados 60 and 80 p.s.i. compression at points 704 and 709 respectively, and intrados 340 and 280 p.s.i. at 704 and 709 respectively.

At 704, the north quarter point, the theoretical stresses on the intrados and extrados are 387 and 240 p.s.i., respectively. The corresponding measured stresses are 250 and 150 p.s.i., respectively in position 115. The theoretical thrust and actual thrust line are at the same elevation 1.5 in. below the arch axis. In the final position at 128, the intrados and extrados stresses are 340 and 60 p.s.i. with the thrust line 4.8 in. below the axis. At this point there is little effect of further rib shortening. This position of the thrust line is favorable for positive live load moments and rib shortening moment from the quarter point to the crown.

Fig. 18 and 19 show stress-crown deflection relations at the quarter points 704 and 709 and are very much the same general relation as for stress-corwn opening.

In Fig. 20 and 21 are shown stresses at point 708, 10 ft. from the crown plotted against crown opening and deflection. With equal jack thrusts on the intrados and extrados, the stresses increase steadily to position 106 for the extrados and 105 for the intrados where they are 170 and 350 p.s.i., respectively. There is no further increase in stress on the extrados to position 109 and an increase of only about 40 p.s.i. on the intrados. Between 109 and 111, where the load is carried on the rings for 20 hours and the wedges are removed, an increased strain corresponding to a stress of about 40 p.s.i. occurs. Between 109 and 110, where this increased strain occurs, there is no change in crown opening and elevation of crown so that this strain is actually plastic change. At 111, where the elevation and jack thrust is the same as at 109, the stress remains practically the same as at 110 and the crown opening is increased about 0.12 in., the plastic change over the period of 20 hours on the rings.

For equal intrados and extrados jack thrusts the extrados stresses remain almost constant over the entire range of crown opening of 1.85 in., while the intrados stress has a range of only 70 p.s.i. However, the stresses are materially changed by the introduction of unequal intrados and extrados pressures. The stresses for the final position are 490 p.s.i. for the intrados and 120 p.s.i. for the extrados.

At position 115, thrust 856,000 lb., the theoretical stresses for the intrados and extrados are 340 and 307 p.s.i., respectively. The thrust line is 0.25 in. below the axis. The measured stresses at position 115 are 400 and 210 p.s.i., respectively. The thrust line is 1.7 in. below the axis. In final position 128, the stresses are 490 and 120 p.s.i. and the thrust line is 3.4 in. below the axis. The thrust line is lifted toward the axis by further shortening and positive live load.

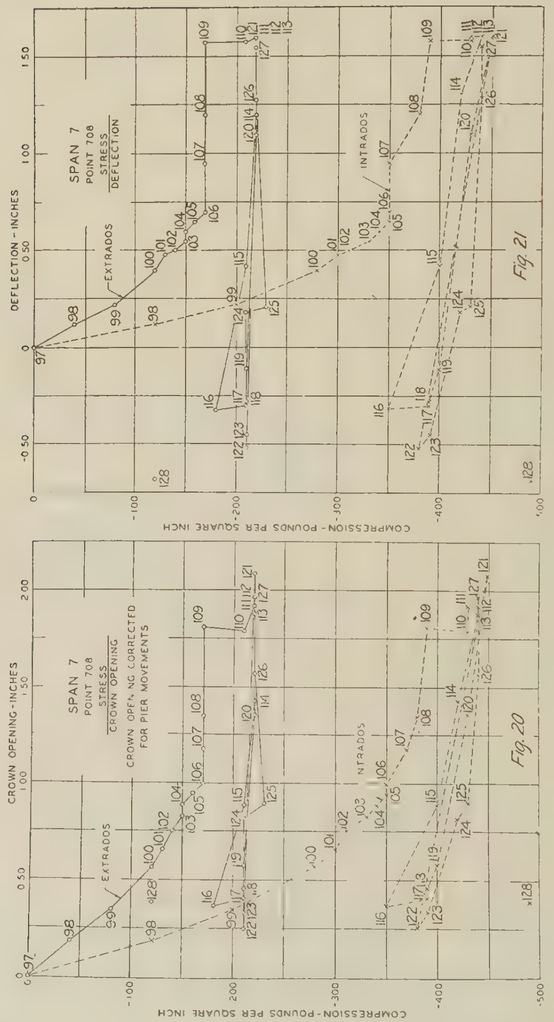

SPAN 7
POINT 708
STRESS
DEFLECTION
DEFLECTION - INCHES
COMPRESSION - POUNDS PER SQUARE INCH
EXTRADOS
INTRADOS
Fig. 21
SPAN 7
POINT 708
STRESS
CROWN OPENING
CROWN OPENING CORRECTED FOR PIER MOVEMENTS
CROWN OPENING - INCHES
COMPRESSION - POUNDS PER SQUARE INCH
EXTRADOS
INTRADOS
Fig. 20

FIG. 20, 21

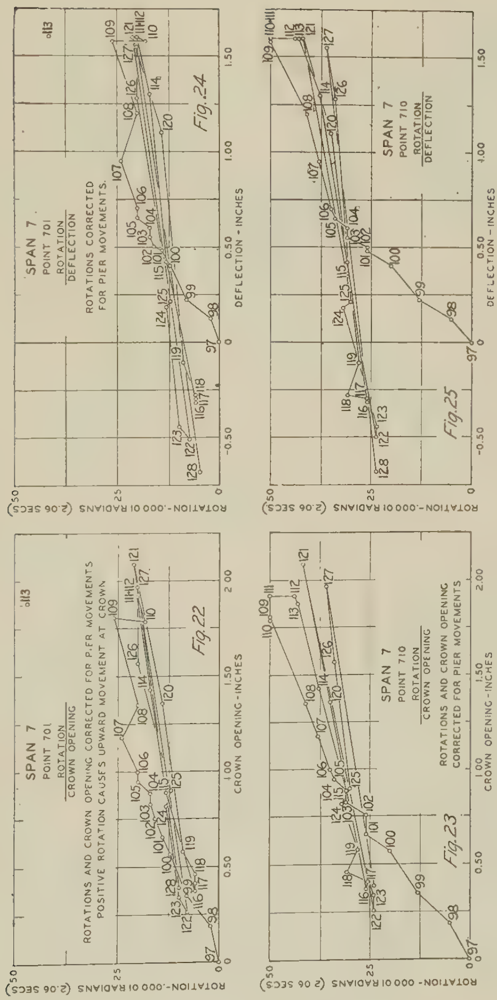


FIG. 22, 23, 24, 25

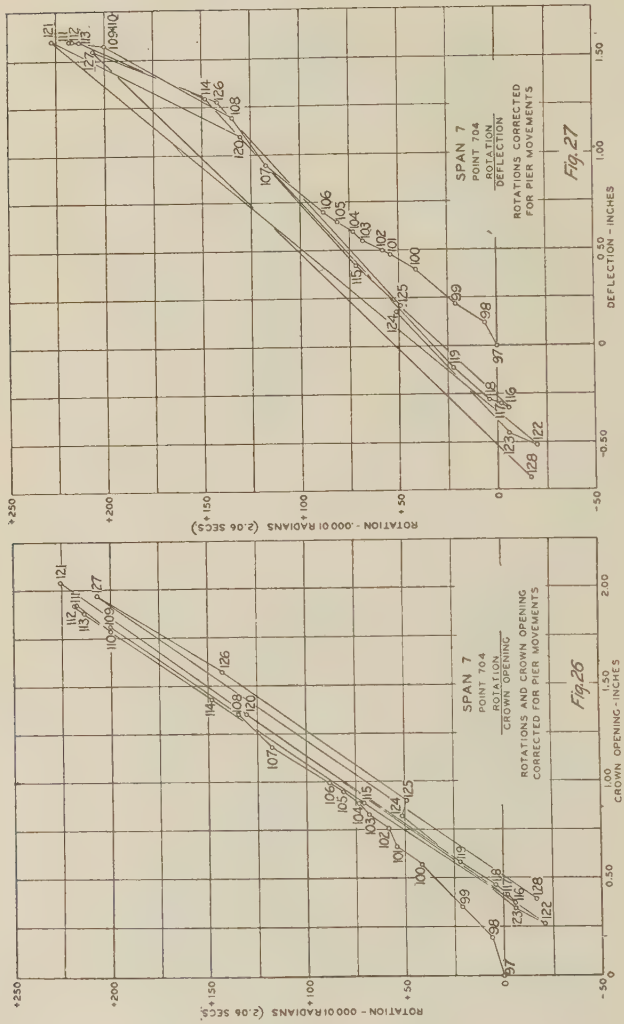
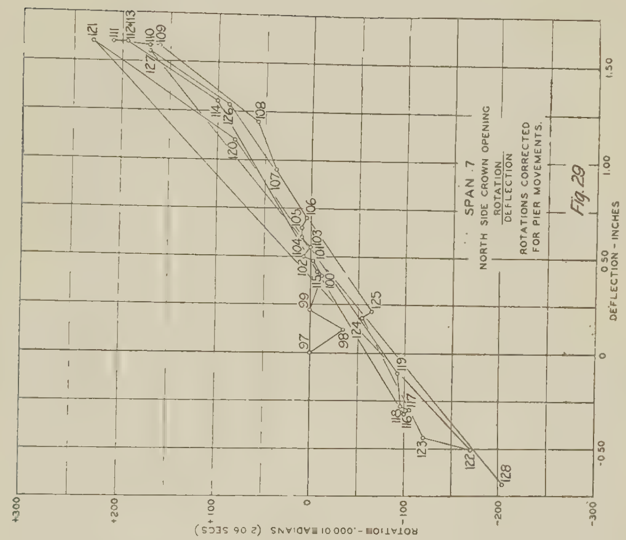

SPAN 7
POINT 704
ROTATION / DEFLECTION
ROTATIONS CORRECTED FOR PIER MOVEMENTS
Fig. 27
DEFLECTION - INCHES
ROTATION - .0001 RADIANS (2.06 SECS.)
SPAN 7
POINT 704
ROTATION / CROWN OPENING
ROTATIONS AND CROWN OPENING CORRECTED FOR PIER MOVEMENTS
Fig. 26
CROWN OPENING - INCHES
ROTATION - .0001 RADIANS (2.06 SECS.)

FIG. 26, 27


SPAN 7
NORTH SIDE CROWN OPENING
ROTATION / DEFLECTION
ROTATIONS CORRECTED FOR PIER MOVEMENTS.
Fig. 29
DEFLECTION - INCHES
ROTATION - .00001 RADIANS (2.06 SECS)

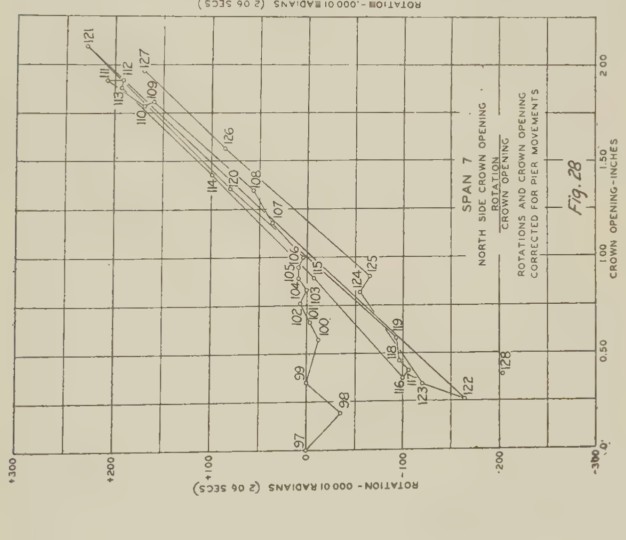

SPAN 7
NORTH SIDE CROWN OPENING
ROTATION / CROWN OPENING
ROTATIONS AND CROWN OPENING CORRECTED FOR PIER MOVEMENTS
Fig. 28
CROWN OPENING-INCHES
ROTATION - .00001 RADIANS (2.06 SECS)

FIG. 28, 29

Fig. 22, 23, 24 and 25 show rotations at the skewbacks against crown opening and deflections. At these points, rotations increased more or less uniformly to position 106 when the rate of increase becomes less. It will be remembered that it was at this point that the stress curves changed slope. It will be seen later that practically no rotation of the joint faces occurred until position 106 was reached. From position 112 to position 128 there is a straight line relation between rotations and crown spread and rotations and crown deflections.

In Fig. 26 and 27, are rotations at the quarter point 704 plotted against crown opening and deflection. After the first increment of load the rotation increases more or less uniformly to 109 although the break between 101 and 102, where the first separation of ribs and centering occurs, is noticeable. A remarkable proportionality of rotation and crown spread is observed. The relation can be closely represented by a straight line. The total range of rotations is about 8.5 minutes of angle practically all of which is positive. It may be pointed out that between 109 and 111 where the thrust was carried on the rings for 20 hours, the rotation increases with the crown spread although no change takes place in the vertical position of the crown.

Fig. 28 and 29 show rotation at the crown in relation to crown spread and deflection. There is practically no rotation of joint faces until position 106 is reached when the rotation begins to increase rapidly. After 106, there is a close proportionality of rotation and crown spread. The total range of rotation here is 13.5 minutes. Again between 109 and 111 there is an increase of rotation.

We have seen that, after the arch was swung entirely free of the centering in position 111 and then lowered to a position where the crown is at the same elevation as in its unstrained position on the centers, there is a total rib shortening of 0.70 in. or .00024 in. per in. An elastic strain of this magnitude would require a unit stress of 960 p.s.i. If we calculate from the telemeter readings at the crown and the jack thrust the modulus of elasticity of the rib we get a value of 4,200,000, a very close agreement with the modulus determined from cylinders. This seems to indicate that the telemeters recorded in this case only elastic strain.

Apparently the major part of the shortening is due to shrinkage which was restrained by the reinforcement and the forms. The reinforcing steel is put in compression and the concrete in tension which produces minute cracks at intervals along the rib. As the jack pressures were applied these cracks were closed. This movement would be reflected in the telemeter readings only in the case where the telemeter crossed a crack. Obviously, in this span none of the telemeters was across a crack because it would have been observed in the preliminary readings before jacking.

Until a more complete study can be made of all the data on the Rogue River bridge, the following tentative conclusions can be drawn:

1. The Freyssinet method of arch construction is simple and effective in eliminating the deleterious effects of rib shortening due to all causes and permits an adjustment of the arch to its most favorable elastic state under combined dead and live loads.

2. This method permits greater economy in arch construction even in moderate span lengths and makes longer spans and smaller rise ratio practicable.

3. The electric telemeter is well adapted to use in connection with the jacks in making the adjustments.

4. A wide range of strains in the ribs can be produced by the jacking process. Within this range, strains can be chosen which will allow for future shortening due to shrinkage and differential temperature changes and which, when combined with live load strains, will reduce the total stresses to a minimum.

5. It is advisable to make a preliminary adjustment on the rib without superstructure and, after the superstructure is placed, make a final adjustment. In this way the effect of superstructure restraint may be definitely determined, a closer adjustment is possible and centers and falsework may be quickly released for re-use.

6. The effect of superstructure restraint, even with almost complete superstructure articulation, is so marked and so difficult to evaluate mathematically that any adjustment of the complete structure should be made with the aid of strain gauge measurements at a number of critical points on the ribs in conjunction with measurements of jack thrusts and joint openings.

Readers are referred to the JOURNAL *for February 1933, for discussion which may develop. Such discussion should reach the Secretary by Dec. 1, 1932.*

Central Concrete Mixing Plant, Puget Sound Navy Yard

BY A. D. HUNTER*

The central concrete mixing plant at the Puget Sound Navy Yard has handled, in a highly satisfactory manner, the heavy load placed upon it by recent Deficiency Act construction projects and the routine construction and maintenance work at the Yard. The present plant is a development from the old central mixing plant in use in the Yard up to 1929. The substitution of a new one-yard mixer and bulk cement handling equipment, gave a considerable increase in efficiency.

The main features of the plant layout include the following:

(a) Storage space for and means of unloading and handling sand and several grades of coarse aggregate.

(b) A main building in which bulk cement is handled and stored and containing the testing apparatus, laboratory, test cylinder curing room, and locker rooms.

(c) A Fuller bulk cement unloader.

(d) A large overhead, three compartment, storage bin for sand and gravel and an overhead storage bin for bulk cement.

(e) An aggregate batcher of the weighing type.

(f) Accurate water control apparatus.

(g) A concrete mixer of 1 cubic yard capacity.

(h) A gravel crusher with conveyor and grading screen.

(i) Equipment and space for extensive pre-casting operations.

Sand and gravel are brought in on barges of approximately 400 cu. yd. capacity. If the plant is operating under light loads, the Yard can supply enough barges; under heavy loads, "foreign" barges from the company supplying the material are in service. Barges are unloaded in an average of four hours by clam-shell buckets on locomotive cranes. Storage space has a section each for sand, paving gravel, and medium gravel and other space for crushing gravel and for crushed stone stock piles. Further storage space, served by the clam-shell cranes is available for about 1400 yds. of sand and 3000 yds. of gravel.

*Lieutenant, (CEC) U. S. N., Puget Sound Navy Yard, Bremerton, Wn.

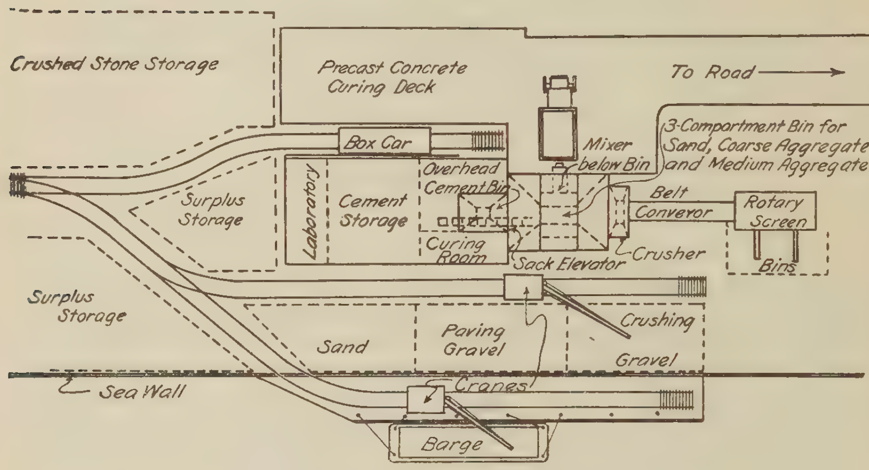


FIG. 1—PLAN PUGET SOUND NAVY YARD CENTRAL MIXING PLANT

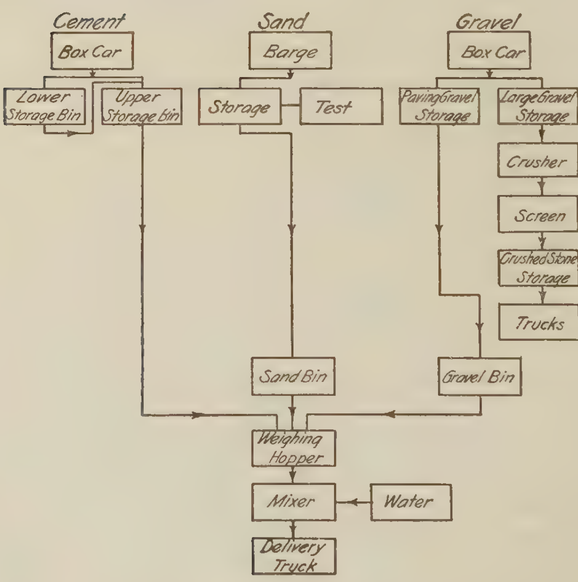


FIG. 2—FLOW SHEET PUGET SOUND NAVY YARD CENTRAL MIXING PLANT

Fig. 3—General view of navy yard central mixing plant

A gravel crushing outfit consists of a motor driven jaw type crusher, capable of crushing 50 cu. yds. of 3 in. gravel in ten hours. The crushed gravel is transported on an electric driven belt conveyor to an electric driven revolving screen which segregates the various sizes. The screen is mounted on a steel support and discharges into ground level bins. The saving effected by the use of the gravel crushing outfit has been considerable. At present gravel is being delivered at $0.725 per cu. yd., f. o. b., contractor's scows at the Navy Yard. The cost of crushing is about $0.20 per cu. yd. Under former conditions it was necessary to pay from $2.50 to $3.00 per cu. yd. for crushed stone of inferior quality. Crushed rock is used for construction and repair of bituminous macadam roads and for railroad track ballast.

Cement comes in bulk via rail ferry and is unloaded directly from box cars to storage bins. The main building or cement shed is of reinforced concrete construction 27 ft. by 54 ft. in plan. Supported on the roof at the eastern end is an overhead cement bin with a capacity of 320 bbls. The large cement storage room has a capacity of 1280 bbls., thus permitting storage of five carloads of cement at one time.

The cement unloader was manufactured by the Fuller Company of Catasaqua, Pa., and with it one man will unload, from a car, 320 bbls. of bulk cement in about three hours. The rated capacity of the unloader is 175 bbls. per hour but due to packing of the cement in transit, this capacity is not obtained. In moving cement which has not been packed during transit, however, the full capacity of the

unloader is realized. The unloader consists of a portable pump which takes the bulk cement from the car by means of a screw conveyor and then forces it by air pressure through a flexible pipe and rigid connections into either the overhead bin above the mixer or into the storage space. An automatic high bin signal when the bin is full, switches the cement flow to the storage space. When the mixer operator observes that the bin supply is getting low, he can, by push button control, change the flow of cement back into the overhead bin. An emergency electric drive sack elevator has been provided for use on rare occasions of failure of bulk equipment.

The mixer, a Rex-28-S machine of one cubic yard rated capacity which on occasions has been operated successfully at one-third above the rated capacity, is directly east of the main building under a steel superstructure which supports the weighing platform and a 135 yard, three compartment, steel bin. Sand, coarse aggregate, and a medium coarse aggregate are kept in these compartments available for the mixer. Each bin has a manually controlled hopper discharging into the weighing batcher.

Originally these bins were built up much higher and a bucket type chain elevator was necessary to convey the materials to the bins. By lowering the bins so that they could be served by a locomotive crane, it was possible to discard the elevator which had been a constant source of trouble and delays due to breakdowns. As a crane was required to load the hopper for the elevator, no additional equipment was required. The lowering of the bins was made possible by the substitution of the one-yard non-tilting mixer for the two-yard tilting mixer formerly used.

Cement is brought to the batcher from the upper cement bin by an electrically driven screw conveyor. The mixer man has complete control of the flow of the various aggregates, cement, and water. The scale dial has movable markers which can be set at desired positions for different mixes. The water is supplied from an accurately graduated tank through a manually operated quick acting valve. The ingredients are mixed for 1½ minutes and the mixer then discharges into the delivery truck.

Apparatus is available for determining the water content of the sand, for drying sand, making standard sieve analyses, and for making accurate weight determinations. A room equipped with thermostatic heat control is used for curing test cylinders. Space is available for the storage of 200 cylinders. At present, all strength testing is done by an outside laboratory.

In some cases bottom dump buckets are carried on the truck and are used to carry the concrete.

Deliveries are made in bottom-dump buckets, on the trucks, when the concrete is to be deposited in a section served by railroad tracks and where locomotive cranes are available to handle the buckets off and on the trucks.

Adequate space and equipment for a large variety of precasting operations are also available. Duct covers, slabs, reinforced concrete piles, warhead storage igloos, retaining wall sections and electric light standards are among the items which have been turned out in large quantities.

Under normal conditions the operating force of the plant consists of one foreman, one crane man, one mixer man, one cement unloader man, one crusher man, and one general utility man, a total of six. When unloading barges an additional crane man and a man on the barge are used. Formerly, when only sacked cement was used, seven additional men were needed, viz; three dumping cement, two unloading, one on the elevator and one cleaning and bundling sacks. This labor item alone amounts to about $30.00 per day saving with the bulk handling equipment.

The maximum capacity of the plant is about 300 cu. yd. of concrete in an 8-hour shift.

Flood lighting allows working three shifts when necessary without loss of efficiency. While concrete operations were in progress on Dry Dock No. 1, an average of 244 cu. yds. of concrete per day was poured over a period of 66 days operation. At the same time, there were several other projects in progress which were supplied by the central mixing plant. The largest single day's run was 620 cubic yds. In the fiscal year 1930-1931 the plant turned out a total of approximately 52,000 cu. yds. of concrete. The gravel crushing outfit went into operation on April 1, 1931, and up to September 15, 1931 it had cruhsed 8,200 cu. yds. of gravel. The quality and uniformity of the concrete being produced by the plant is excellent.

Readers are referred to the JOURNAL *for February 1933, for discussion which may develop. Such discussion should reach the Secretary by Dec. 1, 1932.*

Discussion of Progress Report of Committee 312:

"Plain and Reinforced Concrete Arches"*

Albin L. Gemeny† (by letter)—This report is an ingenious solution of the problem of rib shortening stresses in which the basic assumptions are seldom if ever admissible under actual construction conditions.

It is tacitly assumed by the author that shrinkage and flow occur concurrently and continuously in accordance with laws developed by experimentation on small laboratory specimens of 1:2:4 dry concrete. As a matter of fact, it does not appear that shrinkage occurs to any great extent immediately after placing a mass of concrete of the dimensions of an arch rib and it is further retarded by curing. After the curing period the rate of shrinkage depends largely upon weather conditions which are generally, at the end of a construction season, unfavorable to rapid shrinkage.

At the Rogue River bridge in Oregon[1] the conditions were particularly favorable to rapid shrinkage. There was practically no rain during the construction period and, during the greater part of the period, a steady wind blew. Yet, during the three months following the placing of the ribs, shrinkage was less than one quarter of that in a small cylinder comparable in size to the test specimens which were used by Glanville and Davis in securing the data upon which the author bases his theory. The total shortening due to all causes, shrinkage before decentering and shortening under dead load, was less than one half the shrinkage shortening of the small specimen. At the end of the three months period the rainy season began and consequently it may be expected that the greater part of the shrinkage will occur during the next dry season, from nine months to a year after placing the concrete. The greater part of the flow will have occurred by this time and, therefore, can not be counted upon to relieve bending stresses due to this delayed shrinkage shortening.

The investigations of Glanville and Davis show the following facts in regard to flow:

*"Progress report on the limitations of the theory of elasticity, and the effect of plastic flow, shrinkage, temperature variations and the Freyssinet method of adjustment," by Charles S. Whitney, author-chairman, presented at the 28th Annual Convention, Washington, D. C., March 1-4, 1932. A. C. I. Journal, March 1932, *Proceedings*, Vol. 28, p. 479.

†Senior Structural Engineer, Bureau of Public Roads.

[1]See "Freyssinet Method of Arch Construction Applied to the Rogue River Bridge in Oregon," by Albin L. Gemeny and C. B. McCullough in this Journal issue, p. 57.

(1) The flow of a specimen loaded at the age of three months is approximately one half of the flow of a specimen loaded at the age of one month.

(2) A large part of the total flow occurs during the first six months after loading. These investigators show that as much as 85 per cent of the first year's flow may occur in the first six months.

(3) Flow for a 1:1:2 concrete mixture is approximately only one half that for a 1:2:4 mixture in the first year.

(4) Dry concrete may have a flow as great as 4.5 times that of wet concrete.

With these facts in mind it seems reasonable to conclude that

(1) The greater part of the shrinkage in an arch rib may be delayed from six months to a year after placing the concrete.

(2) Since most of the flow occurs in the first six months it can have but little effect in relieving delayed shrinkage stresses.

(3) Rich mixes which are commonly used in arches are subject to greater shrinkage and much less flow than the 1:2:4 mix upon which the author bases his calculations.

(4) The delayed loss of moisture which may be expected on most construction work again greatly reduces the flow even in the first six months.

(5) Reduced flow in the first six months due to a rich mix and delayed loss of moisture throws considerable doubt upon the wisdom of depending upon this flow to relieve stresses caused by substructure movements.

It is obvious that early decentering of an arch can have no greatly beneficial effect in relieving shrinkage stresses, although it would act to relieve to some extent stresses produced by substructure movements. Contrary to the contention that shrinkage stresses may be neglected, it seems altogether probable that when residual stresses due to rib shortening resulting from flow are added to delayed shrinkage stresses the total residual stress may be in excess of that which would be computed by neglecting the effect of flow.

The Freyssinet method permits progressive adjustment to take care of shortening at any time after the opening of the bridge to traffic. Mr. Freyssinet adjusted an arch two years after construction. The first adjustment indicated that practically no shrinkage occurred in six months following decentering but began to occur rapidly during the following season.

In view of the above considerations, it seems reasonable to conclude that the author is somewhat over-optimistic in depending upon flow to relieve these troublesome deformation stresses and that such dependence would become dangerous in long spans and in spans of low rise ratio.

*Clyde T. Morris** (by letter)—In discussing a paper "A Study of Bending Moments in Columns," by F. E. Richart, at the convention of the Institute, in Chicago in February 1924, the writer spoke as follows:†

> I wish to call attention to a property of concrete which may have some bearing on the stresses that will be developed at the connection of girders to columns, and that is the property of flow, the time factor in adjusting the concrete to the deformations which take place. The deformation of concrete continues for some four or five years before the curves become horizontal, and I think that the dead load moments at these connections perhaps can be greatly reduced if not neglected in the design. This may be one reason why there are fewer failures than might be expected from analyses of existing buildings.

Mr. Whitney, in his report makes a very similar statement:

> Concrete possesses a property which has not been fully appreciated. Its plasticity at early ages makes it more adjustable to permanent strains than it has been considered, and explains why it has given better service in some cases, than could have been expected of a brittle material.

It is this property of plasticity of concrete at early ages, that makes it advantageous to decenter an arch as soon as the concrete has attained sufficient strength safely to support the dead load. In this way stresses in the concrete caused by permanent bending strains, due to dead load, shrinkage, and foundation movements may be greatly reduced. This advantage is largely lost if decentering is delayed.

Due to this property of plasticity of the concrete, the Freyssinet method of adjustment has little advantage if applied at an early age, and may induce serious stresses if it is delayed for a year, as was done by Freyssinet in the three 590 ft. spans at Brest. This adjustment at that age puts bending strains into the rib which the plasticity of the concrete is unable to absorb.

Regarding conclusion 16 of the report, the combined effect of plastic flow and shrinkage, in flat arches, should be compensated for, by constructing the arch with an excess of camber, sufficient to prevent ultimate sagging of the roadway, instead of resorting to the Freyssinet method of adjustment.

The properties of plastic flow and shrinkage of concrete are bound to cause high unit stresses in the steel reinforcement and these stresses in the steel cannot be reduced materially by increasing the percentage of reinforcement. But this need not be a cause for alarm, because steel is a uniform and very reliable material and stresses approaching the elastic limit are not unsafe so long as buckling is prevented, and it is hard to conceive of buckling taking place inside a reinforced concrete member, properly tied transversely.

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*Professor of Civil Engineering, Ohio State University.
†*Proceedings* of the American Concrete Institute, V. 20 (1924) p. 518.

In conclusion 18, Mr. Whitney suggests that welded splices of the reinforcing bars might be advisable where the steel stresses run high. Lap splices of the reinforcing would seem to be better because of the possibility of bond flow relieving some of the high compressive stress in the steel.

W. H. Glanville and F. G. Thomas** (by letter)—Mr. Whitney has made a very thorough survey of existing information relevant to the design of reinforced concrete arches, and has carried the subject a very definite step forward by the development of logical methods of allowing for the flow of concrete under sustained strain. The value of the methods depends primarily upon the truth of equation No. 8 $\left(f_c = \frac{f_o}{e^{E_c C}}\right)$ and Mr. Whitney has stated that it is supported at least approximately by observed facts but lacks full experimental verification. As additional support of the validity of the relation it will be seen, by substitution in equation No. 4, that the case of sustained strain is that of a reinforced concrete column in which the quantity of reinforcement is infinite (i. e. when $p = \alpha$). The experimental evidence that this relation holds for a reinforced concrete column may, therefore, be regarded as additional support for the truth of equation No. 8.

It is known that concrete which has been maintained under load for a period and has flowed under load, will, when the load is released, tend gradually to recover its original length until finally the residual may be considerably less than the maximum flow. It has been suggested that this is due to strain energy stored in the aggregate but, whatever the cause, it would be expected to affect the residual stress when concrete is maintained at constant strain and the stress is gradually reduced by flow. As the stress decreases some gradual recovery of the flow would be expected. Moreover, this effect would be expected to increase as the percentage of steel increases and be greatest for the case of sustained strain (or infinite reinforcement).

Since the publication of Mr. Whitney's paper one or two very simple experiments have been made at the Building Research Station to indicate how far equation No. 8 can be expected to hold for a very simple case. The results have shown surprisingly close agreement with the theory and it would appear from these simple tests that the recovery of the flow under load is much smaller than might be expected.

The tests were made on small 1 x 1 in. mortar beams loaded at two points 6 in. apart on a 24-in. span, as shown in Fig. 1. One specimen

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*Building Research Station of the Department of Scientific and Industrial Research, Garston, England.

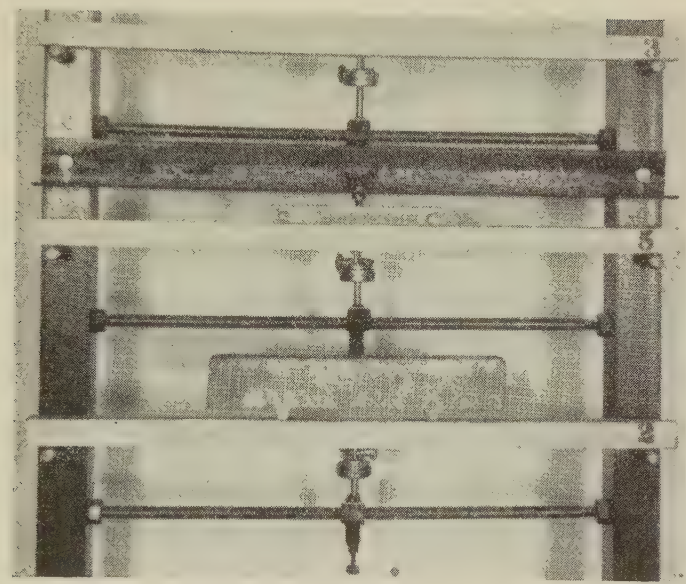

FIG. 1—CREEP TESTS ON CEMENT MORTAR BEAMS
(a) At constant deflection (Beam No. 5)
(b) At constant load (Beam No. 2)

was maintained at a constant deflection by means of a screw working in a rigid framework, and another was maintained under a constant load provided by a lead weight of 15 lb. Deflections were recorded by dial gauges. Periodically the screw on the beam at constant deflection was unscrewed to obtain the residual elastic deflection. A third specimen was used to give shrinkage corrections.

If δ_o = the initial elastic deflection under the load of 15 lb. and therefore the constant deflection,

δ_c = the residual elastic deflection of the beam at constant deflection,

δ_i = the increase in deflection of the beam under constant load, we see that,

δ_o corresponds to f_o

δ_c corresponds to f_c

δ_i corresponds to C

and $1/\delta_o$ corresponds to E_c in Mr. Whitney's equation No. 8.

The equivalent equation is, therefore,

$$\delta_c = \frac{\delta_o}{e^{\delta_i/\delta_o}},$$

and given the results of the test under constant load, i. e. the quantities δ_o and δ_i, it should be possible to estimate the results of the test under constant deflection, i. e. δ_c.

This has been done for two series of tests and the results are shown in Fig. 2 and 3. It will be seen that the agreement of the actual

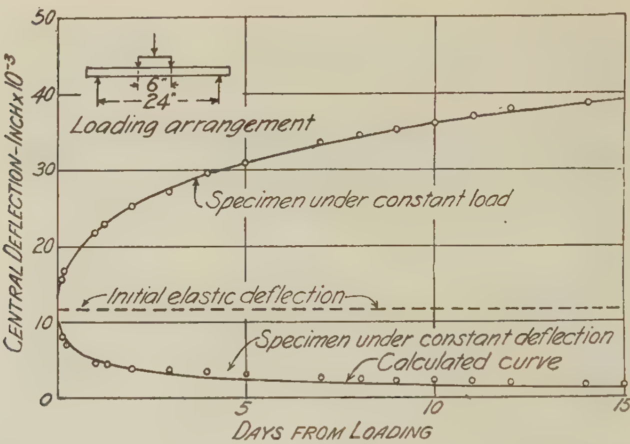

FIG. 2—MOVEMENTS OF CEMENT MORTAR BEAMS UNDER LOAD—SERIES 1

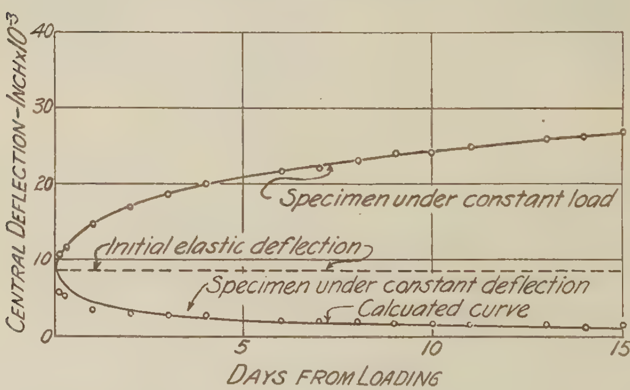

FIG. 3—MOVEMENTS OF CEMENT MORTAR BEAMS UNDER LOAD—SERIES 2

residual elastic deflections with the calculated values is surprisingly good.

With regard to Mr. Whitney's suggested method for calculating shrinkage stresses by the use of residual stress coefficients there is one aspect to which attention should be drawn. The residual stress coefficient method indicates only the ultimate stress condition and does not necessarily show the maximum shrinkage stress that can occur. The difference between the ultimate and maximum values will

generally be small; in one case that has been tested the maximum value which occurred at the age of 10 months is of the order of 10 to 20 per cent greater than the value after two years. This can be shown from the following equation,

$$ds = f_c dc + \frac{df_c}{E_c}$$

which holds when the concrete is held at constant length.

From this equation it is seen that the concrete stress increases only so long as ds is greater than $f_c dc$. The relative rates of shrinkage and creep are thus of fundamental importance in determining whether or not the net effect of shrinkage is an increase or decrease in stress and, therefore, whether the ultimate stress is less than the maximum value.

*Bernard L. Weiner** (by letter)—Mr. Whitney's valuable paper is another contribution toward taking the mystery out of concrete as a structural material. "Practical engineers" will have no use for this paper as they have decided long ago that concrete is an erratic material to be designed only by "practical methods" which is simply another name for slothful, uneconomic, and often even dangerous rule of thumb methods. While it is probably true that it will never be worth while to go into too much refinement in the design of concrete structures, the trouble has never been one of mere accuracy. It is much more fundamental. Until recently, the inherent difference between this material and structural steel had not been recognized. The result has been that engineers have blamed the material instead of their own ignorance.

While the existence of plastic flow has been recognized, its magnitude relative to the elastic deformation, and its importance as a design factor have been underestimated. This paper points the way towards the solution of many puzzling problems.

It has often been stated that while the theory of elasticity as applied to indeterminate structures of concrete will give the correct reactions, the deflections can not be predicted; the reason being that for the solution of the reactions, only relative deformations are necessary, and as long as the elastic deformations are roughly proportional to the true deformations, the results are approximately correct. The deformations computed were only the elastic deflections and hence did not agree with the actual which included plastic flow and the other deformations discussed in this paper.

The writer has read many reports on the tests made on reinforced concrete columns. The results, looked at from the point of view of

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the usual column formula (equation 3) are very confusing. The author's equations 4 to 7 point to the probable explanation of these results. The same thing can be said about the usual practice of designing for temperature stresses. Many specifications allow higher unit stresses when these stresses are included, on the assumption that the computed temperature stresses are dissipated by plastic flow. Also, the temperature range is made rather large to allow for shrinkage, etc. The end of this crude method of design seems in sight.

The writer and others have found difficulty in following discussions on plastic flow because of the lack of a mental picture of its action. The following analogy seems very useful and is given here for those who may have the same difficulty. A spring is mounted on a hydraulic jack which has a leaky piston. The whole device is put into a frame so that a load is applied and the spring compresses. The frame keeps the distance from the bottom of the jack to the top of the spring constant. Since the piston of the jack leaks, the pressure of the spring will eventually force it down slightly, allowing some distance for the spring to expand, thus relieving the pressure. If the jack can travel far enough, all tension in the spring may eventually be relieved. The spring corresponds to the elastic deformation and the jack to the plastic flow. This analogy corresponds to the case where the total deformation is constant and the change takes place from elastic deformation to plastic flow. A similar analogy can be worked out for the other cases discussed.

Whether the theory as developed by Mr. Whitney is finally substantiated or further investigations show corrections to be necessary is of little importance for the moment. An excellent attempt has been made to reduce the various properties exhibited by concrete to a workable mathematical form. The importance of this paper cannot be overestimated. The writer looks forward with a great deal of interest to the further or final reports by the author.

*Gilbert C. Staehle** (by letter)—This report states, in effect, that concrete behaves as an elastic solid under stress due to live (momentary) load, but does not so behave under stress due to (a) dead (sustained) load and (b) forces other than load, i. e., moisture and temperature variations and plastic flow. In other words, its conclusions are that time yield invalidates the elastic theory in its application to arches of plain and reinforced concrete.

Such conclusions are indeed indicated by test data from physical investigations by Davis, Glanville, and others and are supported by Straub's theoretical investigation.

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But, from several phenomena characteristic of concrete of all grades, the writer adduces that it does behave as an elastic solid under load, whether momentary or sustained, in the absence of internal stresses due principally to shrinkage. Conversely, its behavior must be inelastic under either momentary or sustained load if such internal stresses are present. The phenomena referred to are:

(a) Concrete which has been cured in an atmosphere of high relative humidity and not afterwards subjected to drying has a definite proportional limit on a first application of load, although true proportionality between stress and strain may not exist if swelling has occurred. This proportional limit, expressed as a percentage of the ultimate strength, is closely in line with that of steel and other materials classified as elastic, proper weight being given to gradation in quality. (This is a general statement applicable to the majority of modern portland cements.)

(b) The stress-strain graph for concrete which under a first loading has a decided curvature, with subsequent unloading and reloadings and unloadings is, or tends to become, straight; a new proportional limit being established at each new "high" intensity of loading up to a point not far below the ultimate strength.

(c) The modulus of elasticity for loadings subsequent to a first sustained loading approximates closely the original (initial tangent) modulus of elasticity plus an increment due to increase of age for the particular condition of curing or exposure. This is indicated by data from the investigations of Davis and others.

If shrinkage accounts for curvature of the stress-strain graph, as is indicated by the preceding observations, an increased resisting moment or relief to flexural concrete stress by plastic flow is also accounted for.

Note that there is no need to distinguish between shrinkage and plastic flow since plastic flow inevitably follows the application of a first load to shrunken concrete.

Considering the applicability of these phenomena to arches of plain and reinforced concrete, it appears that:

(a) The modulus of elasticity, which is defined as the constant ratio between unit stress and unit strain below the proportional limit, applies to calculations for direct stress due to dead and live load, dead and live load rib or ring-shortening, and abutment movements.

(b) A reduced concrete stress-strain ratio, which in the report is given in the form $R = 1/e + c$, corresponding to the design age, applies to calculations for direct stress due to residual strain (shrinkage and plastic flow and temperature variations, also moisture variations in the opposite sense to shrinkage if considered).

(c) A variable concrete stress-strain ratio, corresponding to the design age, applies to flexural stress due to any cause, considering residual strain, but because hygrometric conditions may preclude or offset such residual deformations the flexural concrete stress due to dead and live load, dead and live load rib or ring-shortening, and abutment movements should be calculated on the basis of a constant stress-strain ratio.

The elastic theory thus applies to all stresses except those due to shrinkage and plastic flow and temperature variations (the inclusion

of temperature variations, lacking comprehensive and reliable test data, is to be recommended), if correction is made for the increment of steel stress concomitant to flexural relief of the concrete by possible residual strains. It applies also to the calculation of crown deflections with similar exceptions.

Except for arches of low rise and for special cases, precise values of c are not indispensable, since the stresses to which it applies are small relatively to the sum of all stresses. Such values may be taken directly, for corresponding conditions of sustained or momentary loading and exposure, from published test data, due weight being given to quality of concrete and its constituent materials. Where conditions warrant it more precise values may be determined by experiment if the particular materials of which the concrete is to be made can be determined in advance, its quality of manufacture and curing rigidly controlled, and critical hygrometric conditions forecast with reasonable accuracy. The more extended and thorough the curing the more dependable such values will be.

The writer does not agree that 4,000,000 is a proper value of E_c for general use in arch design. The lower strength concretes will hardly attain such a high value even under the most favorable curing and exposure, at the critical design age. Low strength concretes are slow to attain a stable modulus, that is, a modulus which increases at a low and nearly constant rate with further aging. High strength concretes, particularly those in the high early strength class, on the contrary attain such a stable modulus quickly. Since an accurate determination of horizontal and vertical deflections, of steel stress and of the moment of inertia for reinforced concrete, is dependent on an accurate estimate of the modulus of elasticity, it does not seem logical to adopt a single value for all grades of concrete and without reference to age and exposure. If dependable test data for the particular concrete are not available, the writer suggests that E_c be assumed as equal to $1{,}500{,}000 + 600\ f'_c$, where f'_c is the ultimate unit strength at the age considered. For multiple arches on elastic piers, particularly if one arch is to be centered at a time, for yielding foundation material, and in general where possible deformations are likely to impose severe stresses, the value E_c should be based on reliable test data applicable to the particular conditions and concrete.

The advantages to be secured from increased plasticity by early decentering are debatable. Early decentering is a likely cause of unequal and unpredictable abutment movements, therefor no advantage would seem to inhere in this respect. Flexural relief of the concrete will be more marked while the concrete is plastic, that is, while

its yield point is low, but if the decentering is delayed the deformations causing bending will be less, so the net result would appear to be the same or approximately so, or is apt to be in either direction. Also, as the report states, such plasticity results in appreciable initial stress in the steel, which may not be desirable. In the case of high strength concretes immediate plastic effects should be negligible, especially if the curing is adequate, since plastic yielding is less marked the higher the quality of the concrete.

AUTHOR'S CLOSURE

Charles S. Whitney (by letter)—Mr. Glanville's experimental verification of equation 8 is particularly interesting. It appears that Equation 8 will give the residual stress with reasonable accuracy if the flow can be predicted. It is obvious that the rates observed on small dry specimens cannot be expected to occur in large bodies but there is no reason to think that the fundamental relations are different.

Mr. Glanville has correctly stated that ultimate shrinkage stress may not be the maximum shrinkage stress. The time at which the maximum stress is produced will depend on the relative rates of shrinkage and flow. If the total amounts of shrinkage and flow at any time are known, the formulas may be used to determine the shrinkage stress at that time. It is interesting to note that if the shrinkage proceeds more rapidly than the flow, the increase in stress resulting has some compensating effect by increasing the rate of flow.

Mr. Gemeny's discussion has not increased the writer's confidence in the Freyssinet theory which is based on elastic behavior and neglects the effect of flow on the stresses. The report has pointed out the difficulty in predicting the rates of flow and shrinkage from experiments on small specimens and has recommended observations on actual bridges. The values suggested in the report were selected because they appear to be consistent with results of observations on bridges. There is furthermore no evidence that the fundamental laws developed by experimentation are not applicable to large arches. Experimentation shows for instance that the same climatic conditions which retard shrinkage also retard the rate of flow, so the assumption that shrinkage and flow occur concurrently is probably not so seriously in error in so far as the ultimate stress condition is concerned.

The investigations of Glanville and Davis referred to by Mr. Gemeny were made under controlled conditions and do not necessarily lead to his conclusion that most of the flow occurs in the first six months when conditions are such that the greater part of the shrinkage may

be delayed from six months to a year. It is clear that both flow and shrinkage will occur and that the flow will affect the stress condition. If the flow and shrinkage curves could be predicted, the method presented by the writer would give the stresses at any age with reasonable accuracy. It is obviously impossible to predict the exact shape of the flow and shrinkage curves and it is necessary to assume approximate values from experience with actual bridges and with laboratory specimens.

The small amount of shrinkage in the Rogue River arches up to the time of de-centering, amounting to less than one-quarter of the shrinkage of cylinders, was to be expected because in addition to the reduction of shrinkage due to the moisture in the larger mass, the restraint of the forms would eliminate most of the shrinkage occurring while the concrete was very plastic. The small shortening during decentering is further proof that the effect of high setting temperatures is eliminated by flow while the arch is restrained by the forms.

The report suggests for discussion flow and shrinkage values for 1:2:4 concrete. These do not apply to richer mixes nor to rapid setting cements. As Mr. Gemeny points out, richer mixes are subject to less flow and greater shrinkage but the modulus of elasticity is also greater. This has something of a compensating effect because the proportionate stress release is a function of the product of the flow multiplied by the modulus of elasticity according to equation 8. The value of this product is affected by so many variables that it cannot be too carefully chosen. Further study of the modern finely ground and rapid setting cements is especially needed.

Mr. Gemeny says "Contrary to the contention that shrinkage stresses may be neglected, it seems altogether probable that when residual stresses due to rib shortening resulting from flow are added to delayed shrinkage stresses the total residual stress may be in excess of that which would be computed by neglecting the effect of flow." The writer does not contend that shrinkage stresses may be neglected, but he does contend that the stresses in reinforcing steel due to rib shortening resulting from flow added to shrinkage stresses are greatly in excess of the stresses computed by neglecting the effect of flow. The latter method does not even give approximate results.

The bridge referred to by Mr. Gemeny which was adjusted by Freyssinet two years after construction consisted of extremely flat three-hinged arches which were jacked up to remove unsightly sagging. According to Freyssinet's report,[1] the bridge was put into service at

[1]Le Pont de Villeneuve-sur-Lot, Perfectionnements dans la Construction des Grandes Voutes. E. Freyssinet. *Genie Civil*, July 30, August 6, 13, 1921.

the beginning of 1911. At that time, the shortening amounted to 1×10^{-4} and Freyssinet applied a correction of 4×10^{-4}. By the middle of the summer of 1911, the shortening practically balanced the correction. In 1913, the total shortening amounted to 8×10^{-4} causing a sag of 4 to 5 inches. The arches were then jacked and corrected. The shortening increased slightly in 1914 and there has been no appreciable increase since then. According to these figures, about one half of the total shrinkage and flow occurred during the first six months after decentering.

Dependence should not be placed on flow to eliminate deformation stresses, but it is certain that the effect of flow must be considered in a determination of the magnitude of such stresses. A careful determination is very important in the case of long span arches. The Freyssinet method cannot be justified by any theoretical analysis which neglects the effect of flow. Further study is needed to ascertain whether or not its efficiency is great enough to justify its use in any particular case.

Professor Morris suggests that lap splices may be better than welded splices in the reinforcing steel because of the possibility of bond flow relieving some of the high compressive stress in the steel. Arch reinforcing rods are so long that bond flow could not be expected to relieve the stress for the full length and there would appear to be considerable disadvantage in having it relieved only in the neighborhood of the splices with resulting higher local stresses in the concrete. A welded butt or even a short lap splice would also avoid doubling the amount of reinforcing steel for an appreciable length of arch which must result in an indeterminate stress condition at the splice. In order to avoid unconformities it would seem best to approach as nearly as possible the condition of continuous reinforcement without splices.

Mr. Weiner has given a graphic description which helps to visualize the effect of plastic flow in relieving stresses which would otherwise exist. The writer cannot follow Mr. Staehle's discussion of the application of the elastic theory and he does not agree that there is no need to distinguish between shrinkage and plastic flow. The recognition of such need is the principle difference between the writer's theory and that of Mr. Freyssinet. While they may be primarily due to the same physical action in the concrete, the shrinkage according to the writer's conception is that part of the inelastic strain which is independent of stress and the plastic flow is that part which is dependent on the stress. They must be separated if the effect of load is to be determined. The theory of elasticity obviously cannot be applied without modification to cases of inelastic strain.

The writer wishes to thank those who have discussed the report. He believes that the theory presented is at least pointing in the right direction and that it can be clarified by further research.

Discussion of Report of Committee 902:

"PROPOSED SPECIFICATIONS FOR CONCRETE PAVEMENT IN MUNICIPALITIES"*

R. W. Crum†: I have one general criticism that I would like to make, not only in this case but in others. I noticed in looking through this specification a number of instances in which the committee had gone into considerable detail in writing specifications on matters covered by existing standards or tentative standards of this and other authoritative organizations. It seems to me that the societies would all work to better effect if, in such instances as these, a new specification would use the existing standards. If the committee is at variance with those standards, steps should be taken to get in touch with the originators of the standards and endeavor to reconcile the differences so that the various organizations and the various committees of this Institute could work to better advantage and so that specifications as finally promulgated would have the support of every one interested in them as well as that of the one group.

President Hollister: Is there any further discussion? Is it understood, Mr. Goldbeck, that you desire this specification referred to the Aggregates Committee?

A. T. Goldbeck‡: Well, I think that will be done automatically by the chairman of this particular committee; he will unquestionably refer this specification to Mr. Clemmer, the chairman of the Institute's Committee 201, Aggregate Specifications. The aggregates specification as originally written into this pavement specification was the 1929 report of Committee 201[1]. That is the latest specification published by the Aggregates committee and revisions that occur here are the revisions made by Committee 902, Concrete Pavement Standards, so that we *did* consider the original specification of the Aggregates committee.

*A. C. I. JOURNAL, March 1932, *Proceedings*, Vol. 28, p. 453, presented by A. T. Goldbeck, member of the Committee together with the discussion at the 28th Annual Convention, Washington, D. C., March 1-4, 1932.

†Director, Highway Research Board, Washington, D. C.

‡Mr. Goldbeck is a member of both Institute Committees 201 and 902 and presented the report of the latter committee in the absence of its chairman, F. C. Lang.

[1] "Tentative Purchase Specifications for Concrete Aggregates" (E5-A-29T) *Proceedings*, American Concrete Institute, 1929, Vol. 25, p. 657.

Discussion of a paper by J. C. Pearson and E. M. Brickett:

"Studies of High-Pressure Steam Curing"*

F. O. Anderegg† (by letter)—Results with dolomitic aggregates in line with those reported by Messrs. Pearson and Brickett have been obtained by the writer. Several series of bars 2 x 2 x 8 in. of 1:4 mixes were made up and cured under different temperature conditions going as high as 212° F. After curing, the bars were stored in the air of the laboratory for six days and then immersed in water for 24 hours. The increases in length ranged from 0.05 to 0.04 per cent approximately, the ones cured at higher temperatures showing slightly smaller volume changes. The bars were then broken in flexure at 28 days giving moduli of rupture ranging on the average from 513 to 423 p.s.i. The flexural strengths decreased with the time and temperature of curing.

Dalton G. Miller‡ (by letter)—There is no question that the curing of concrete products in high pressure steam has great possibilities, considered from a number of angles, but it is well to bear in mind that not all concretes, made of sound aggregates, cured in high pressure steam can be expected to develop strengths in excess of, or even equal to, strengths developed by normal curing.

The effects on strength and resistance to sulfate waters of curing concrete in water vapor at temperatures between 100 and 350° F. following 24 hours in moist room at room temperatures of 68 to 75° F., when compared with tests of check specimens cured continuously in water at room temperatures, following 24 hours in moist closet, were summarized in a paper published in 1930.[1] In these steam-cured groups there were some 12,000 cylinders which were made between 1921 and 1929 of portland cements from 14 different mills. Briefly, the effect on compressive strength was reported:

> Solely from the standpoint of strength, little or nothing is gained by curing concrete in water vapor at a temperature much above 155° F. and little is gained, even at this temperature, by prolonging the curing period beyond 48 hours.
>
> Concrete after 12 to 24 hours in water vapor at temperatures between 100 and 350° F., followed by storage in dry air at room temperatures, has a compressive

*A. C. I. Journal, April 1932; *Proceedings*, Vol. 28, p. 537.
†Consulting Specialist on building materials, Pittsburgh, Pa.
‡Bureau of Agricultural Engineering, U. S. Department of Agriculture.
[1]Dalton G. Miller, "Strength and Resistance to Sulfate Waters of Concrete Cured in Water Vapor at Temperatures Between 100 and 350° F." *Proceedings*, A. S. T. M., Vol. 30, Part II, p. 635 (1930).

strength at 7 days not greatly different from that of 7-day-concrete cured continuously in water at room temperatures.

As was noted in the 1930 A. S. T. M. paper, these conclusions are at variance with results of the 1907-1908 tests reported by Wig[2] and it is evident are also at variance with the conclusions of Messrs. Pearson and Brickett. In lieu of any other explanation to account for these observed differences of strengths of concrete cured at these high temperatures, it seems reasonable to assume the cause to have been differences in the aggregates used; probably, or at least possibly, some differences in quantity, fineness, or chemical activity of the silica present. In the manufacture of sand-lime brick, strengths upward to 8,000 p.s.i. are obtained using steam pressures of 125 to 150 p.s.i. It is evident that portland cement concrete cured at these high temperatures therefore reasonably could be expected to be stronger than that normally cured, particularly at earlier ages, if conditions were such that some of the free lime of the cement, or lime of some form in the sand, could readily combine with silica, of the sand, active at these high temperatures. That some concretes cured at high temperatures do not develop strengths in excess of that normally cured may be indicative of insufficient active silica in the sand to bring this about.

*T. Thorvaldson** (by letter)—Studies on the effects of curing portland cement mortars and concrete in steam under pressure have been in progress at the University of Saskatchewan during the last eight years as a part of an investigation of the action of sulfate waters on cement and concrete. This work was initiated by a committee of the Engineering Institute of Canada, under the chairmanship of Dean C. J. Mackenzie and has been carried on with the financial support of the National Research Council of Canada and other public organizations. The results which have been obtained enable one to explain some of the observations recorded in the above paper and to answer some of the questions at the end of the paper.

It has been found[1] that the first effect of curing in steam under pressure is to decrease the tensile and compressive strength of the specimen. This is quickly followed by the well known increase in strength. For short periods of steam-curing (1 to 12 hours) there is usually an acceleration of the increase in strength as the temperature of the saturated steam rises up to 200 to 225° C., for longer periods of steam curing there is an optimum temperature for obtaining high

[2]R. J. Wig, "The Effect of High-Pressure Steam on the Crushing Strength of Portland-Cement Mortar and Concrete." Technologic Paper No. 5, U. S. Bureau of Standards, 1911; also *Proceedings*. A. S. T. M., Vol. XI, p. 580 (1911).

*Professor of Chemistry, University of Saskatchewan, Saskatoon, Canada.

[1]Thorvaldson, Vigfusson and Wolochow. *Can. J. Research*, Vol. 1, 370-374 (1929).

strengths which is usually below 200°C. The rate of increase of strength and this optimum temperature also vary with the composition of the cement and the time of damp-curing which the specimen has been subjected to before exposure to steam. Thorvaldson and Shelton[2] found that during the first few hours of the steam-curing of a sand mortar made from a portland cement high in tricalcium silicate, crystals of calcium hydroxide appear in the specimen, followed on longer steam-curing by the disappearance of these crystals and the gradual formation of another crystalline product. Later work by V. A. Vigfusson and the writer has shown that this second crystalline product is a hydrated calcium silicate formed mainly by the interaction of the lime, liberated by the hydration of the tricalcium silicate in the cement with the silica of the aggregate, and that this same crystalline product is formed during the manufacture of sand-lime brick. It is the opinion of the writer that the increase in strength produced by the curing of portland cement mortars and concrete in steam under pressure is mainly due to this reaction, although it has been shown that other reactions mentioned below probably contribute materially to the result.

This throws light on several observations noted and discussed by the authors of the paper, (pages 545-549) for instance, the effect of varying the cement content, the low increase in strength of neat cement specimens, and the effect of using a calcareous instead of a siliceous aggregate. These observations are in agreement with results previously obtained in the writer's laboratory. It is evident that when the aggregate is calcereous or when dealing with specimens of neat cement, there can be no development of strength due to the formation of hydrated calcium silicate by reaction between the lime liberated during hydration and the aggregate. The effect of the cement content is also in accordance with what one would expect on the basis of a reaction between the lime and the silica which takes place at the interface. For the richer mixes the limiting factor is the amount of surface of the aggregate while for lean mixes the limiting factor is the quantity of cement so that the strength developed per pound of cement per cubic foot increases with decreasing cement content and tends to reach a maximum for lean mixes at least at the higher temperatures of steam-curing where the reaction is fairly complete. This reaction between free lime and the aggregate probably increases in importance as the time of the preliminary damp-curing is increased.

[2]*Can. J. Research*, Vol. 1, 148-154 (1929).

There are other reactions which play an important part in the curing of portland cement concrete in steam under pressure. The formation of the hexahydrate of tricalcium aluminate[3] in steam under pressure in preference to the higher anisotropic hydrate formed at room temperature may play a part in the initial stages. Norman B. Keevil, working in this laboratory, has shown that both tricalcium silicate and β-dicalcium silicate in steam under pressure form crystalline hydrates without any liberation of free lime. This process is probably only second in importance to the reaction between the free lime and silica in the development of strength of portland cement mortar on steam curing and may be of primary importance in concrete made from cements high in dicalcium silicate, or specimens exposed to steam after very short periods of preliminary damp curing. Further, Douglas T. Mather, also working in this laboratory, has shown that the compound $4CaO. Al_2O_3 . Fe_2O_3$, which is probably present in portland cement, hydrolyses very slowly in steam giving the hexahydrate of tricalcium aluminate, anhydrous ferric oxide and free lime, and that the velocity of this reaction increases with the temperature of the saturated steam. This reaction may be, at least in part, the cause of the falling off in strength which generally takes place on steam-curing of specimens for long intervals of time, and which at the higher pressures may occur within 24 hours with a second increase in strength following this.

To sum up: The loss in strength at the beginning of steam-curing of portland cement mortars or concrete is probably due to changes in the crystalline form of hydrated tricalcium aluminate and to physical stresses during the expansion of the specimen. The increase in strength which rapidly follows this is due to the formation of a crystalline hydrated calcium silicate by reaction of free lime with the siliceous aggregate and to the direct formation of crystalline hydrates of the silicates of calcium, the formation of a hydrate of tricalcium silicate being mainly responsible for the early increase of strength while the hydration of dicalcium silicate is responsible for the later more gradual increase. Later losses in strength, especially at higher pressures of steam, are probably due to the hydrolysis of the compound $4CaO. Al_2O_3. Fe_2O_3$, and possibly to secondary changes in the calcium hydrosilicates.

The question whether the high strengths obtained by steam-curing are permanent in character is an interesting one. Tests made in this laboratory, extending over from three to eight years, indicate

[3]Thorvaldson and Grace. *Can. J. Research*, Vol. 1, 36, 1929 Thorvaldson, Grace and Vigfusson. *Can. J. Research*, Vol. 1, 201, 1929.

that steam-cured portland cement mortars stored in water retain their strength, and that specimens which have been subjected to short periods of steam-curing continue to increase slowly in strength and ultimately attain as high strengths as specimens cured in water. The enormous increase in the stability of steam-cured mortars and concrete in sulphate waters has been discussed elsewhere.[4]

AUTHORS' CLOSURE

The authors appreciate the value of the foregoing discussion as supplementing the information contained in their paper. They are indebted particularly to Professor Thorvaldson, who has been able from his own researches to explain a number of things that were not understood. For example, it was natural to conclude that the cement itself developed more strength in the accelerated curing than it did in normal curing, but a review of the data shows that the remarkably high strengths were obtained only when siliceous aggregates were used. On the other hand, the authors were convinced that the calcareous aggregates were positively damaging, but this has not been shown conclusively, because the effect on the cement is not certainly indicated by the behavior of the neat cement specimens that were included in the program.

Obviously some further studies are needed for a proper evaluation of the effect of high-temperature steam curing on *cements*, if a satisfactory understanding is to be had of the part played by each of the constituents of the concrete in this type of curing.

[4]Thorvaldson, Vigfusson and Wolochow. *Can. J. Research*, Vol. 1, 359, 1929.

Discussion of a paper by R. E. Toms and J. W. Johnson:

"The Design and Construction of the Mount Vernon Memorial Highway"*

CONVENTION DISCUSSION

Chairman Hollister—The Institute has not, at any time, had the good fortune to have for its records so complete a report as this of design and construction of what might be classed in all respects a model highway. The Institute members will have an opportunity this afternoon of seeing this highway. Current reports that the trip this afternoon would by-pass a portion of this work because it is not yet open to traffic, are incorrect, because special permission has been obtained to pass the busses over this route from end to end. At Mount Vernon there will be an opportunity of one hour given to inspect and visit the shrine of the father of this country. Perhaps no visit to Washington is complete without such a trip, and certainly there is no place in this country of historic value which so completely represents the spirit of the days of the founders of this country as does Mount Vernon.

A. E. Lindau—It is only fair that some one in the audience express our appreciation of these papers and not leave it entirely to the chairman. I am not surprised that we have had little discussion. It seems every conceivable contingency has been provided for, and there is not very much room for discussion, but I wish to express personal appreciation of the manner in which this work has been conceived, planned and executed.

D. A. Abrams—I have been reading some of the letters George Washington wrote about the time he made his trip from Mt. Vernon to become the first president of the United States. I could not help but think of these letters in looking at the pictures of this excellent highway as it stands now. In George Washington's time highways were very scarce. In these letters George Washington speaks of his trip, the tremendous difficulties that were involved. He started about six weeks in advance in getting horses and carriages and drivers to make the trip. Mrs. Washington was thrown out of the carriage more than once, so I think that gives us a little picture of the advance in highway construction.

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*Presented at the 28th Annual Convention, Washington, D. C., March 1-4, 1932; A. C. I. Journal, April 1932; *Proceedings*, Vol. 28, p. 563.

Discussion of a paper by C. H. Jumper:

"Tests of Integral and Surface Waterproofings for Concrete"*

BY M. W. MEYER†

Admittedly the matter of waterproofing concrete is most difficult to test in the laboratory. Under its carefully controlled conditions, impervious concrete can be produced without the use of admixtures or surface treatments—but admittedly, also, it is neither practical nor economical in the field to attempt to duplicate these laboratory conditions. The whole development of waterproofing materials and their extended and increasing use in concreting is the most forceful proof that under average field conditions impervious concrete is *not* produced without their assistance.

Confining our attention to the work on integral waterproofings, exception must be taken to the general statement in the summary of results (p. 229): "the addition of calcium chloride . . . did not materially reduce its permeability or absorption." Fig. 2 shows a wide variation in results of this particular group as compared with the standard. It is unfair to class the products of manufacturers which produce concretes which "at the end of one year were impermeable" with others showing definitely greater permeability than the standard. Mathematically, at least, the former have decreased the permeability 100 per cent. The wide variation in results produced by wirebrushing and drying of the samples are too significant to be brushed aside with an all-inclusive generalization that the permeability was not materially reduced. Such statements produce the impression on contractor and the architect that admixtures for waterproofing of concrete are worthless, whereas the fact is clearly shown by the results published in the paper that some commercial products are of distinct value not only in waterproofing but in increasing the compressive strength of the resultant concrete.

*A. C. I. Journal, Dec. 1931; *Proceedings* Vol. 28, p. 209; discussion continued from April 1932; Vol. 28, p. 585.

†Anti-Hydro Waterproofing Co., Newark, N. J.

The same criticism holds for the "Resume and Conclusions." We read (p. 240) "Hence, since concrete can be so made without the use of waterproofings, they would seem to be needless," followed by a "but" paragraph in which hesitancy is the dominant tone, as exemplified by "Doubtlessly some of the integral agents"—and "some *might* react," and "If the integral waterproofings *could* reduce its permeability . . ." There is altogether too little attempt to inform the public that the results show that there are bona-fide waterproofing materials for concrete.

Exception is taken to the impression given by Mr. Jumper that waterproofings and admixtures are unnecessary. There are too many irregular conditions that present themselves on every concrete job which cannot be copied nor looked for in the laboratory and which can be greatly benefited by a proper use of a waterproofing of merit.

The tests conducted by the Bureau of Standards show decided advantage in the increased impermeability produced by some of the integral waterproofings. It is also true that the use of waterproofings and admixtures is most helpful under field conditions. It is generally known and nationally advertised that some manufacturers of waterproofing products offer a field service to insure the proper use of their materials. The engineers and service men specializing in this field work, at the same time check up on the existing field conditions and advise the best methods for pumping, drainage, mixing and placing concrete. This field service plus the use of integral waterproofing permits the manufacturers to furnish a bonafide guarantee for 25 years on the maintenance of impermeable concrete. Some of the waterproofings in Mr. Jumper's report are the same products tested by the Bureau of Standards and reported 20 years ago in Technologic Paper 3, and the 25-year guarantee is based on more than 25 years of satisfactory performance.

It is suggested that when the Bureau of Standards publishes its final report, there be no general statement based on the average results of all or any group of waterproofing compounds. Such general statements might not represent the test result. The public is not interested in the average results, but is interested in obtaining better concrete. If any waterproofings or admixtures produce these results, then the public should have the Bureau's findings.

BY MAURICE G. ROUX*

1. In the absorption tests made by Mr. Jumper, successively immersing and drying cylinders already hardened and dried to a constant

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weight in an oven at 65° C., the results obtained are part permeability and part hygroscopicity.

The tests show that a concrete in which soap alone or soap mixed with hydrated lime, has been incorporated, has less absorption but a higher permeability than the standard: the conclusion therefore is that the treatment reduces the hygroscopicity. A waterproofed concrete will then get wet more slowly than the standard. It would be interesting to find out if the waterproofed concrete can get as wet as the standard, provided it be given enough time, because from that time on the water will enter the waterproofed concrete more quickly than the standard, as it has been proved to be more permeable. For that reason, it would be good to carry the tests on a longer period than three days, perhaps one month or even one year if necessary.

2. It would also be interesting to extend the comparative tests of integrally waterproofed concrete with standard concrete to the field itself or reproduce the conditions of the field in the laboratory. It is of common knowledge for instance that when a mortar is made with the addition of a waterproofing agent (either incorporated in the cement or added at the time of mixing) the material becoming water repellent, the mix must be worked longer and a proper mixing of the cement, the aggregates and the water is much more difficult to obtain.

It would also be good to examine carefully what effect the waterproofing has on the bond strength of a concrete, for it would be of little use to make a concrete of less absorption if at the same time the water could easily seep between the concrete and the bricks or the stones, as some field tests seem to indicate.

3. After a considerable number of tests of various concretes with and without waterproofing agents—tests carried on for a long time, C. M. L. Rengade has also come to the conclusion that the standard concrete (300 kilos of cement per cubic meter) when well made and properly laid is waterproof in practice.

For a basis upon which the use of integral waterproofing agents in concrete might be justified and to have the advantage of a little reduction of the absorption obtained at the expense of the permeability, Mr. Jumper found it necessary to use a mixture so lean that it would never be specified or tolerated on any well supervised job.

BY OTTO GRAF*

The results of these tests are interesting and deserve wide attention. In the concrete tested, admixtures on the whole caused only minor improvement. Regarding these results, it should be borne in mind that the mortar part of the concrete was of such coarse grading as is

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seldom used in Germany.[1] Under these circumstances there must be large voids in the mortar.

Our experience suggests that admixtures for improving the impermeability of concrete are more effective in closing fine voids, such as found in the mortars used in this country. We have also found rich mixes (without admixtures) to be permeable. For that reason it would have been of great value if the investigations had been with concrete of finer grading and with richer mixes, because such concrete is of greater practical importance than that used for the tests.

The admixtures which undergo chemical reactions with the cement demand enough cement in the concrete; different admixtures also react very differently with various kinds of cement.

*L. A. Perry** (by letter)—The author has added materially to current information regarding these, so-called, waterproofer-admixtures. Perhaps the greatest value of this paper is in the analyses of these substances, and it is too bad Mr. Jumper did not see his way clear to identify the materials by their trade names. He is to be complimented upon his statement that the .79 water-ratio was less permeable than the .72 ratio. It takes some courage to put this in writing, because it is contrary to the popular, orthodox theory. The writer has found, in similar tests, the .75 ratio much less permeable than .65 where the paste content was within the range of structural mixes. When this unwelcome fact first asserted itself, the test series were re-run twice—with the same results. Since then, the writer has been able to explain, to his own satisfaction at least, the causes of certain observed effects otherwise incompatible with accepted theories. It seems that the w/c strength curve parallels impermeability performance for only a portion of its range—after which the curves diverge sharply. It will be interesting for some research institution to locat this parting of the ways and find out where we go from here.

It is to be expected that discussion will drag out and dust off the shop-worn reference to "laboratory conditions" or "research technique" as being disassociated from field conditions. Practically all of our knowledge which guides, or should guide, our actual construction has originated in or been collected and distributed by the laboratory. When the conclusion of such research has been found misleading, it was so because the investigator started out to prove a conclusion rather than to reach one. The engineer of tender years reads a bulletin on the harmful effects of excessive mixing water and then proceeds to make concrete so dry that the dust almost blows off of it;

[1]"Der Aufbau des Mörtels und des Betons," by Graf, 3rd Edition, Berlin 1930.
*Consulting Engineer, Seattle, Wn.

if he had but read carefully! the same bulletin confined itself to "workable mixes."

It would seem that the use of a 1:2:4 mix in this investigation would have been more nearly in line with average conditions; then possibly 40 lb. hydrostatic pressure would have been used. But at least some of the materials investigated are sold under a policy that encourages lean mixtures as justification for the existence of the admixture; so if these materials have suffered by such an examination, they have been paid in their own coin.

The writer thinks it would have served the purpose better to compare these mixes under conditions of fixed consistency rather that fixed water-ratio. The general conclusion would be but little different. Some of the powders would have been less permeable and possibly also would have lost some in compressive strength.

In the conduct of a commercial pre-mixed concrete operation, the writer has provided, as a shock absorber for sales promotion, a simple, reliable means of measuring the permeability of mixes with or without admixtures. The latch string is always out to any admixture producer. Up to date there has not been any rush—and no one of the very few materials examined was identified as God's gift to the concrete industry.

The writer thinks that anything (admixtures included) which improves workability will, by this virtue, enhance the water-tightness as well as other desirable properties of concrete, but he is skeptical of the inherent waterproofing qualities of these substances.

Perhaps the cement manufacturer can be looked to for cements of better integral sealing qualities. This may be a "let George do it" attitude but on the other hand, he should be glad to keep novices and opportunists from monkeying with his chemistry. It is just possible that a ground-in pozzuolanic material would be serviceable, since it combines with much of the free lime otherwise removed in solution by the flow of water through pores. It would probably also minimize shrinkage by the attendant crystal formation and limit the annoying compressive stresses set up in steel by shrinkage. It is conceded that this material has the effect of retarding hardening, and in these days we demand "pick up" and acceleration in cements just as we do in automobiles; so it appears that a pozzuolanic addition would require a compensating accelerator.

If the cement manufacturer finds the production and identity of these special cements so burdensome as to be reflected in prohibitive prices, it may even be found that the pozzuolanic material may be supplied during mixing and that some simple commodity like calcium

chloride will serve as a compensator. Such materials would not be admixtures as the term is frequently understood, because they would be bought in bulk, subject to test and analysis, and their use approved by cement manufacturers. If such practice ever becomes common, the responsible central mixing plant operator will enjoy a new field of service, because such plants are better equipped to control such practice than the average field mixing plant.

AUTHOR'S CLOSURE

C. H. Jumper†—The maker of Integral Compound No. 35 has called our attention to the fact that in addition to the components which are shown to be present in his product in Table 1, there are also present "resinous gums." A further examination of the sample shows that it contains 0.39 per cent carbon by weight. This would indicate that there is present less than one per cent of a resinous substance. It was not intended that the compositions of the compounds given in the several tables should be considered other than indicating the general nature of the major constituents of the compounds. But when a producer feels that a constituent though present in very small quantities is of major importance, it is a pleasure to bring this to the attention of the public.

†Bureau of Standards, Washington, D. C.

Discussion of paper by R. E. Copeland and A. G. Timms:

"EFFECT OF MORTAR STRENGTH AND STRENGTH OF UNIT ON THE STRENGTH OF CONCRETE MASONRY WALLS"*

F. O. Anderegg† (by letter)—In view of the marked variation among limes, a more complete description would be desirable. Was the lime dolomitic or high in calcium? Was a putty used or a dry hydrate? What putty yield was obtained from 100 lbs. of quick lime and how much water was required to make up the putty?

Mr. Copeland—The lime used in the mortar mixtures was a high calcium, dry hydrate.

*Presented at the 28th Annual Convention, Washington, D. C., March 1-4, 1932; A. C. I. JOURNAL, April 1932; *Proceedings* Vol. 28, p. 551.
†Consulting specialist on building materials, Pittsburgh, Pa.

The Strength of Concrete Masonary Walls After Standard Fire Exposure*

BY C. A. MENZEL†

SUMMARY AND CONCLUSIONS

THIS paper presents data on the load-carrying ability of concrete masonry walls, made with units of widely varying characteristics, both during and after exposure to standard fire test conditions. Comparative data on similar walls not exposed to fire are included. The investigation comprised tests on more than 200 walls 5½ ft. wide, 6 ft. high, and 4, 8, and 12 in. thick. The following conclusions summarize the principal findings:

(1) The compressive strength of concrete masonry walls, tested both without exposure to fire and after exposure to fire, was directly proportional to the original compressive strength of the units. This linear relationship was obtained with walls laid up with units of a wide range in composition, design and strength.

(2) The strength of walls, tested without exposure to fire and constructed of units of a given design and strength, was independent of the type of aggregate, depended to some extent on the type of mortar, but depended mainly on the type of mortar joints and character of mortar bedding. After exposure to fire, the wall strength was influenced to a more marked degree by the type of aggregate than by the type of mortar but to an even greater extent by the type of mortar joints and character of mortar bedding.

(3) Closely similar strengths were obtained from walls laid up with units of a given strength with portland cement-lime mortars ranging from 1:1:6 to 1:0.15:3. When the cement content of the mortar was reduced below that of a 1:1:6 mix there resulted a decrease in wall strength which was approximately proportional to the decrease in the cement content of the mortar. These statements apply to walls exposed to fire as well as to unexposed walls.

(4) The strength of walls plastered on either the exposed face or on both faces was appreciably higher after fire exposure than that of

*Subsequent to the presentation of this paper in its original form at the Institute's 28th Annual Convention, Washington, D. C., Mar. 1-4, 1932, revisions have been made by the author to include additional data.

†Associate Engineer, Research Laboratory, Portland Cement Assn., Chicago, Ill.

similar unplastered walls exposed for the same or for shorter periods.

(5) No outstanding advantage was discernable in wall strength after fire exposure for one design of unit over another in tests of walls of the same thickness laid up with units of different design, but comparable as to the proportion of core area, the proportion of net area bedded, and strength (gross area).

(6) An outstanding feature of the investigation was the substantial load-carrying ability and safety exhibited by the walls before, during, and after severe fire exposure. This was repeatedly demonstrated notwithstanding the many variables of composition and design of units and of type of mortar and workmanship employed in the construction of the walls.

(7) The various relationships established by the tests provide basic information for the manufacture of concrete masonry units from a wide range of available materials, and for their assembly into walls which will meet the most exacting strength requirements of regulatory bodies.

INTRODUCTION

For the last three years the Research Laboratory of the Portland Cement Association has been engaged in a comprehensive investigation planned to establish the underlying principles governing the fire resistance and load-carrying properties of concrete masonry. This paper deals only with that phase of the investigation which relates to the strength of the walls during and after exposure to fire and of similar walls not exposed to fire.

SCOPE OF TESTS

The tests were arranged to study the relative influence of the various factors of composition, form, and assembly of the masonry units on the performance of the walls when subjected to the standard fire endurance and load tests. The following variables, among others, were studied: (1) Grading of aggregate, (2) cement content, (3) type of aggregate, (4) design of unit, (5) type of mortar, (6) workmanship in laying up walls, and (7) application of plaster.

The tests were scheduled in groups to bring out the effect of each variable over a wide range of variation in that factor. A total of 215 walls were constructed of which 165 were subjected to the fire endurance test and tested for strength after exposure to fire. Fifty walls were tested for strength without exposure to fire.

EQUIPMENT FOR FIRE AND LOAD TESTS

The equipment was designed specifically for the conduct of fire tests in accordance with Standard Specifications for Fire Tests of Building

Construction and Materials.[1] Both because of limitations in the space available for the test equipment and the excessive cost of an investigation of this magnitude, walls about 5½ ft. wide and 6 ft. high with a total area of about 33 sq. ft. were employed instead of walls 9 ft. high and 100 sq. ft. in area required by these specifications. A complete description of the equipment will be found in one earlier paper by the writer.[2]

Briefly the furnace consists of a shallow vertical combustion chamber with back, sides and roof of portland-cement concrete about 12 in. thick made with crushed fire brick as fine and coarse aggregate. The test wall is carried in a movable frame which closes the front of the furnace. Four hydraulic jacks (capacity of 400,000-lb.) mounted in the movable frame furnish the loading equipment. The heat is supplied by a bank of 27 gas burners passing through the back wall. The burners are staggered to promote a good distribution of the fire over the exposed surface of the test wall, and the draft is controlled to give a quiet, bathing, luminous fire, free from smoke.

Measurement of Temperature, Expansion and Deflection.[3] The furnace fire is manually controlled. By a system of sensitive recording thermocouples the temperatures can be noted and made to follow the standard furnace control curve to attain 1700° F. at 1 hour and 2300° F. at 8 hours exposure. Changes in vertical dimensions of the walls during and after exposure to load and fire are measured by the Ames gages and deflections by measurement from a weighted steel wire stretched about 4 in. from the unexposed face near its vertical center line. The wall is free to expand and deflect in a horizontal direction but is restrained vertically by keeping the hydraulic jacks balanced at the desired working load during and after exposure to fire.

TEST PROCEDURE

The various steps in the routine test procedure were as follows:

(a) *Preparation of Aggregate.*—The fine and coarse aggregates were examined as to their general appearance, grading and other physical properties, and mineral composition. When necessary they were dried and screened to the desired sizes.

The aggregates used were the following:

1. Siliceous sand and gravel (A), containing about 95 per cent silica in the form of flint and chert.
2. Siliceous sand and gravel (B), containing about 85 per cent silica as quartz, and very little calcareous material.
3. Calcareous sand and gravel, containing about 40 per cent calcium carbonate, 30 per cent magnesium carbonate, and less than 15 per cent quartz.
4. Crushed limestone (dolomitic).
5. Crushed common brick.

[1] Tentative American Standard A2-1926.

[2] "Tests of the Fire Resistance and Stability of Walls of Concrete Masonry Units," C. A. Menzel, Proc. Am. Soc. Testing Materials, V. 31, Part 2, p. 607, 1931.

[3] The earlier paper (see Note 2) should be consulted for complete details of these measurements.

6. Crushed firebrick.
7. Haydite., a light, porous burned shale material.
8. Air-cooled blast furnace slag.
9. Soft coal cinders (typical power plant cinders) containing about 12 per cent combustible material.
10. Coke breeze (coke screenings from dust to ⅜-in.)

(b) *Manufacture and Testing of Individual Concrete Units.* All units were made in the laboratory with a block machine of standard make. They were molded by the tamped process from carefully proportioned weight mixtures.

The units were "moist cured" for 5 days at a temperature of 70° F. and in a saturated, foggy atmosphere obtained by atomizing nozzles, and were then stored 9 days in ordinary air of the laboratory. They were laid up into test walls at age of 2 weeks.

The compressive strength of the units was determined at 5, 28 and 60 days. The rate of moisture loss and the absorption were also determined.

(c) *Construction and Aging of Test Wall.* When two weeks old, the units were laid up into test walls. Each panel was built upon a "loading beam" of reinforced concrete 4 in. thick resting on a steel plate 2 in. thick, which distributed the thrust exerted by the four hydraulic jacks to the wall after installation in the test frame.

Except in certain special tests, the mortar used was a 1:3 cement—Elgin sand mixture with an addition of 15 per cent hydrated lime by volume of cement. This 1:3 volume mix was based on dry, rodded sand and corresponds approximately to a 1:4 mix proportioned in the field with damp, loose sand. In general, only as much mortar was employed in the horizontal joints of the block and tile as was required for the proper bedding of the face shells and intermediate parallel webs, if any. This method of bedding may be designated as "good" and is illustrated for the 12 types of walls shown in Fig. 4. The ends of adjacent units were in general bonded by hollow vertical mortar joints consisting of two strips of mortar extending inward 1½ to 2 in. from each face of the wall, leaving an air space between the strips. Both vertical and horizontal mortar joints averaged ¼ to ⅜-in. thick.

All walls were aged at least 45 days during the heating season and 60 days during the summer months so as to approach as nearly as practicable the moisture content of interior walls in service in heated buildings.

The test walls were installed in the movable steel frame from three to five days before exposure to fire. The top bearing edge of the wall was buttered with mortar and caused to bear against the concrete lintel forming the insulation protecting the upper horizontal member of the test frame from the fire.

(d) *Loading of Walls before Fire Exposure.* Just prior to fire exposure, a load of 240 p.s.i. of gross wall area was gradually applied to each wall in approximately equal increments, observations being made of deflections and of the vertical deformations. Soon after these data were recorded, the load was reduced to 80 p.s.i. gross area, the the amount required to be carried during the fire endurance test.*

(e) *Intensity and Period of Fire Exposure.* The wall, loaded to 80 p.s.i. gross area, was exposed on one side to the fire, controlled in accordance with the standard time-temperature curve. The fire exposure period was generally 1¼ hours for 4-in. walls, either 3 or 3½ hours for the 8-in. walls and 5 to 6½ hours for 12-in. walls. In some tests, however, 4-in. walls were exposed as long as 2½ hours, 8-in. walls for 6½ hours, and 12-in. (13-in.) walls for 9 hours.

*This load is prescribed for walls of hollow masonry units laid in portland cement mortar in "Recommended Minimum Requirements for Masonry Wall Construction," June 26, 1924, prepared by the Building Code Committee of the Dept. of Commerce.

(*f*) *Observations During Test and Cooling Period.* Throughout each test, observations were made regarding the character of the fire and its control, the condition of the exposed side as observed through mica windows in the furnace, and all phenomena pertinent to the performance of the wall as a fire retardant, with special reference to stability, heat insulation, and flame passage. At the end of the fire exposure period the wall was withdrawn from the furnace and allowed to cool for 24 hours, frequent readings being taken of temperatures within the wall and at unexposed surface and of deflections and changes in dimensions. All walls remained under their normal working load during cooling.

(*g*) *Loading of Wall to Failure.* Twenty hours after the fire endurance test, when the wall had practically cooled to room temperature, it was gradually loaded to failure. Measurements of bowing and changes of dimension of the wall were made at each load increment.

DISCUSSION OF RESULTS

The significant results from the various groups of tests are presented by graphs and tables which bring out the influence of each factor studied upon the strength of the walls. Each figure is accompanied by brief notes giving pertinent information relative to the units and the wall assembly.

Basis for Evaluating Performance of Walls

In harmony with the scope of this paper the performance of the various test walls is compared principally on the basis of their load-carrying ability after fire exposure. Where possible, the load-carrying ability of the walls after fire exposure is compared with that of similar walls not exposed to fire. No comparisons of wall performance are made in this paper from the standpoint of the other principal criterion —the fire endurance period. By *fire endurance period* is meant the period during which the test walls sustained a working load of 80 p.s.i. of gross sectional area under standard fire exposure, without transmission of flame, hot gases, or high temperatures to the unexposed side as defined by the standard fire test specifications. As only one of the walls failed under load, and none by the transmission of flame and gases hot enough to ignite ordinary combustible materials, the fire endurance periods were determined by the temperature rise on the unexposed face. The requirements of the standards are that the average temperature rise (above initial temperature of wall) of the unexposed face shall not exceed 250° F. and that the maximum rise at any point where temperature measurements are taken, shall not exceed 325°F.

In all but a few of the tests the average temperature rise of 250° F. determined the fire endurance period rather than the maximum permissible rise of 325° F. at any individual point. In all except one case the exposure to fire was continued *beyond* the time at which the per-

missible temperature rise had been attained, being terminated at the next half-hour period. This made possible the comparison of wall strengths at comparable fire exposure periods.

Ultimate Strength Tests after Fire Exposure

The Standard Specifications do not require that bearing walls subjected to fire endurance tests carry loads greater than their working loads after fire exposure. However, ability to carry greater loads is a very desirable property giving evidence of the safety of walls during and after fire exposure and of the extent of the fire damage. The relative load-carrying ability of the various walls was therefore obtained both in terms of ultimate strengths after fire exposure and the ratio of ultimate wall strength to the original strength of the individual units. The latter method was used most as it compensated for variations in the original strength of the units.

EFFECT OF COMPOSITION OF UNIT ON THE STRENGTH OF WALLS

Tables 1 and 2 contain data on the strength of blocks, and on the strengths after fire exposure of 32 walls 8 in. thick (comparable as to type of mortar joints) built with 3-oval-core block units of a wide range in type and grading of aggregate, in cement content and strength.

TABLE 1—STRENGTH OF UNITS AND WALLS FOR DIFFERENT GRADINGS OF AGGREGATE

Walls 8 in. thick made with 3-oval-core block.
Description of aggregates used are given in (a) under "Test Procedure."

Fineness Modulus	Mix by Volume	Cement Content		Average Weight of Air-Dry Block, Lb.	Ultimate Strength, p.s.i. Gross Area		Ratio of Wall Strength to Original Strength of Unit, Per Cent
		Blocks per Sack	Lb. per Block		Block before Fire Exposure	Wall after Fire Exposure Indicated	
Calcareous Sand and Gravel Aggregate							
4.50	1:7.7	21.2	4.43	49.9	1860	450 (3 hr.)	24.6
4.00	1:7.6	21.4	4.39	48.5	1100	205 (3 hr.)	18.6
3.50	1:7.8	22.4	4.20	46.6	790	240 (3 hr.)	30.4
3.00	1:8.2	23.4	4.01	44.3	705	160 (3 hr.)	22.7
2.00	1:7.2	21.4	4.39	41.2	500	80 (3 hr.)	16.0
Highly Siliceous Sand and Gravel Aggregate "A"							
4.25	1:7.5	21.2	4.43	46.4	1810	480 (3 hr.)	25
3.50	1:7.5	21.8	4.31	45.2	1190	240 (3 hr.)	20
2.46	1:7.5	21.6	4.35	42.5	675	100 (3 hr.)	15
Limestone Aggregate							
4.60	1:7.4	21.8	4.31	41.1	800	300 (3 hr.)	37
4.40	1:8.3	21.6	4.35	46.0	1030	Over 240[a] (3 hr.)	..
3.50	1:7.9	21.7	4.33	49.0	1580	530 (3 hr.)	33
2.83	1:7.7	21.7	4.33	47.2	1170	470 (3 hr.)	40
2.16	1:7.5	21.6	4.35	46.8	1100	460 (3 hr.)	42
Haydite Aggregate							
4.75	1:7.0	21.4	4.39	23.8	480	145 (3½ hr.)	30
4.25	1:7.0	21.1	4.45	25.6	680	273 (3½ hr.)	40
3.75	1:7.0	21.3	4.41	27.2	890	324 (3½ hr.)	36
3.00	1:7.0	21.2	4.43	28.2	930	333 (3½ hr.)	36
2.07	1:7.3	21.0	4.47	28.5	840	273 (3½ hr.)	32

[a]Ultimate strength of wall not determined.

TABLE 2—STRENGTH OF UNITS AND WALLS FOR DIFFERENT CEMENT CONTENTS

Walls 8 in. thick made with 3-oval-core blocks.
Description of aggregates used are given in (a) under "Test Procedure".

Fineness Modulus	Mix by Volume	Cement Content: Blocks per Sack	Cement Content: Lb. per Block	Average Weight of Air-Dry Block, Lb.	Ultimate Strength, p.s.i. Gross Area: Block before Fire Exposure	Ultimate Strength, p.s.i. Gross Area: Wall after Fire Exposure Indicated	Ratio of Wall Strength to Original Strength of Unit, Per Cent
				Calcareous Sand and Gravel Aggregate			
4.50	1:14.0	40.0	2.35	46.4	920	270 (3 hr.)	29
4.50	1:11.0	30.0	3.13	48.5	1160	310 (3 hr.)	27
4.50	1:7.0	19.0	4.95	51.1	2440	560 (3 hr.)	23
4.50	1:5.0	14.3	6.58	51.6	2300	550 (3 hr.)	24
4.50	1:4.0	11.1	8.45	52.4	3350	615 (3 hr.)	18
4.50	1:3.0	9.3	10.10	52.4	3550	750 (3 hr.)	21
				Highly Siliceous Sand and Gravel Aggregate "A"			
4.25	1:14.0	38.5	2.44	44.5	725	[a]	[a]
4.25	1:7.5	21.2	4.42	46.4	1810	480 (3 hr.)	26
4.25	1:3.0	9.3	10.10	49.6	4075	855 (3 hr.)	21
				Haydite Aggregate			
3.25	1:10.0	29.1	3.23	26.8	750	256 (3½ hr.)	34
3.25	1:8.0	23.6	3.98	27.8	1000	445 (3½ hr.)	44
3.25	1:7.0	21.1	4.45	28.2	1120	358 (3½ hr.)	32
3.25	1:5.0	15.9	6.15	30.0	1575	580 (3½ hr.)	37
3.25	1:3.0	9.8	9.60	32.8	2100	770 (4 hr.)	37

[a]This wall failed at 93 minutes of fire exposure under the working load of 80 p.s.i. of gross area and was withdrawn from the furnace at 94 minutes.

The curves in Fig. 1, based on the data in Table 1, show the large effect of variations in grading of aggregate on the compressive strength of the blocks.

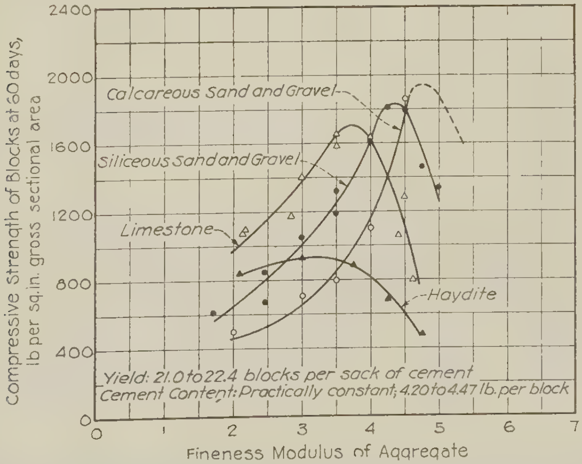

FIG. 1—EFFECT OF GRADING OF AGGREGATE ON THE COMPRESSIVE STRENGTH OF AIR-DRY 8 BY 8 BY 16-IN. 3-OVAL-CORE BLOCKS

Relations between strength of block and cement content, based on the data in Table 2 are given in Fig. 2. The strength of blocks made with each type of aggregate was for all practical purposes directly proportional to the cement content. The relation for blocks made with the two sand and gravel aggregates is represented by a single graph which also illustrates the great range in the strength of the units (725 to 4075 p.s.i.) used in laying up the walls in this group.

Relation between Strength of Wall and Original Strength of Unit

The effect of these large variations in strength of units (produced either by varying the grading or cement content) on the strength of wall after fire exposure is indicated by comparing the strength values

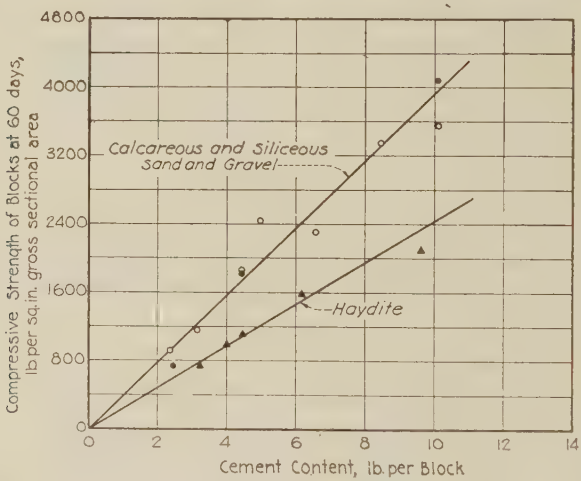

FIG. 2—EFFECT OF CEMENT CONTENT ON THE COMPRESSIVE STRENGTH OF AIR-DRY 8 BY 8 BY 16-IN. 3-OVAL-CORE BLOCKS

in Tables 1 and 2. The relations between wall and unit strength are brought out more clearly by the various diagrams in Fig. 3 which show separately the tests made with the different types of aggregate. Other graphs based on the strength of comparable walls tested without exposure to fire are included in the diagram for each type of aggregate. There are included in Fig. 3 the results of some miscellaneous tests on comparable walls which for sake of brevity are not reported in Tables 1 and 2.

It will be seen from Fig. 3 that the compressive strength of the walls tested both without exposure to fire and after exposure to fire bears a straight-line relationship to the original compressive strength of the

units. The establishment of this direct proportionality between strength of wall and unit was one of the most significant developments of the strength tests.

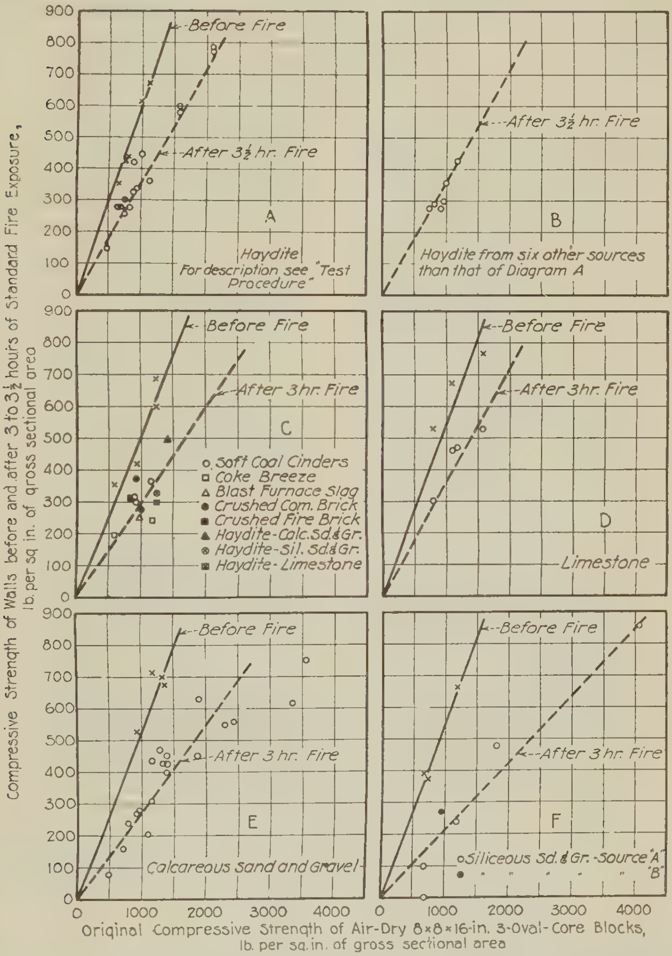

FIG. 3—RELATIONS BETWEEN ORIGINAL STRENGTH OF BLOCKS MADE WITH DIFFERENT TYPES OF AGGREGATE AND STRENGTH OF 8-IN. WALLS AFTER 3 TO 3½ HR. FIRE EXPOSURE

In general blocks made with various gradings of aggregate and with a wide range in cement content.
Walls laid up with 1:3 portland cement mortar plus 15 per cent hydrated lime by volume of cement.
Blocks bedded in mortar applied at face shells only as shown in Diagram 3 of Fig. 4.
Walls exposed to fire allowed to cool to room temperature for 24 hr. before loading to failure.

All of the results in the six diagrams of Fig. 3 are plotted in Diagram A of Fig. 5. While there is some scattering of the points representing the tests after fire exposure, it is seen that in each case the results of the entire group are well represented by a single line. These composite graphs indicate an average ratio of strength of wall to original strength of unit of 30 per cent for walls exposed to fire and 55 per cent for walls not exposed to fire.

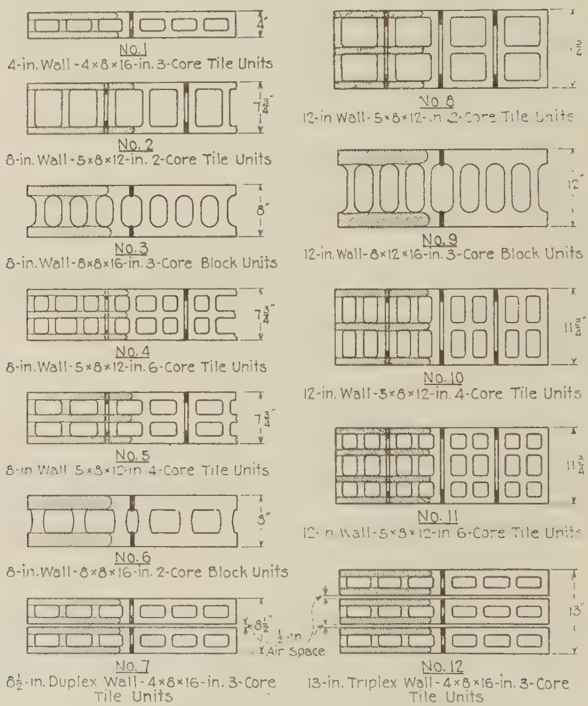

Fig. 4—Details of various types of walls

Comparison of Types of Aggregates

The diagrams in Fig. 3 also afford a means of comparing the several types of aggregates as to their effect upon the strength of walls. The graphs for walls not exposed to fire indicate that for a given strength of unit practically equivalent wall strengths are obtained with all of the different types of aggregate. The ratio of strength of wall to strength of unit varied but slightly from the average value of 55 per cent.

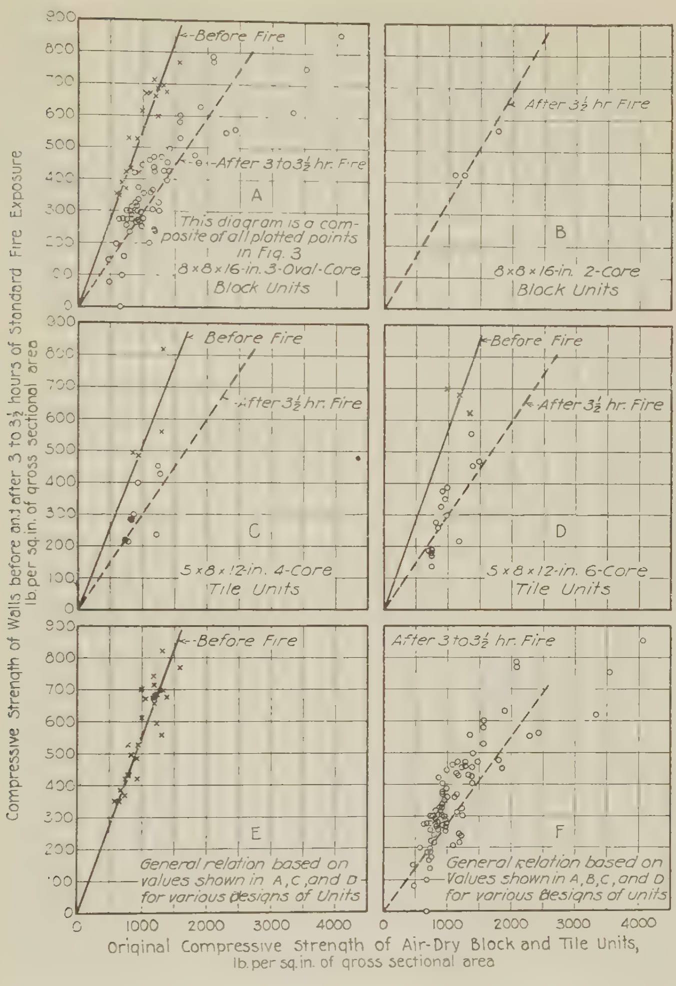

FIG. 5—GENERAL RELATIONS BETWEEN ORIGINAL STRENGTH OF UNITS OF VARIOUS DESIGNS AND STRENGTH OF 8-IN. WALLS BEFORE AND AFTER 3 TO 3½ HR. FIRE EXPOSURE

Plotted values include all tests made with different types and gradings of aggregate and with different cement contents.

Walls laid up with 1:3 portland cement mortar plus 15 per cent hydrated lime by volume of cement. Blocks bedded in mortar applied at face shells only as shown in diagrams 3 and 6 of Fig. 4.

Tile units bedded in mortar applied at face shells and central web as shown in diagrams 4 and 5 of Fig. 4.

Walls exposed to fire allowed to cool to room temperature for 24 hr. before loading to failure.

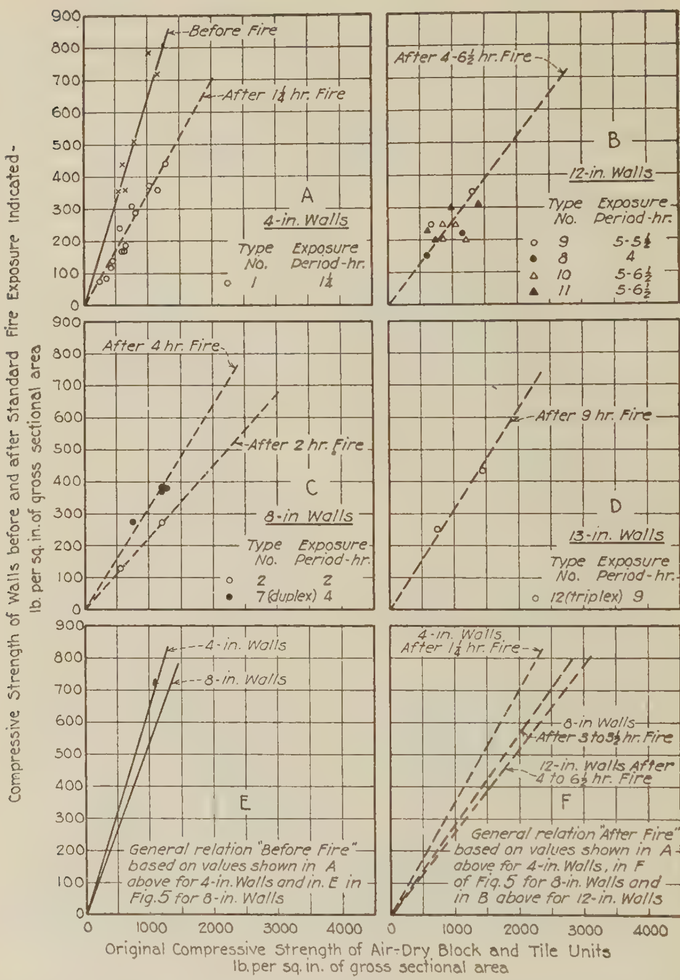

FIG. 6—GENERAL RELATIONS BETWEEN ORIGINAL STRENGTH OF UNITS OF VARIOUS DESIGNS AND STRENGTH OF 4, 8, AND 12-IN. WALLS OF VARIOUS TYPES BEFORE AND AFTER FIRE ENDURANCE TESTS RANGING FROM 1¼ TO 9 HR.

Plotted values include all tests made on walls 4, 8 and 12-in. thick of the various types shown and numbered in Fig. 4 of units made with different types and gradings of aggregate and with different cement contents.

Walls laid up with 1:3 portland cement mortar plus 15 per cent hydrated lime by volume of cement. Mortar was applied at the bearing and end surfaces of the units as indicated by the shaded areas in the diagram for each type of wall in Fig. 4.

Walls exposed to fire allowed to cool to room temperature for 24 hr. before loading to failure.

The effect of type of aggregate on wall strength after fire exposure, however, is more pronounced as may be seen from a comparison of the following ratios of wall strength to strength of unit obtained from the respective graphs for each type: Haydite and limestone 36 per cent; cinders, blast-furnace slag, crushed common brick, crushed fire brick, and coke breeze 30 per cent; calcareous sand and gravel 28 per cent; highly siliceous sand and gravel 22 per cent. On the basis of these values, 8-in. walls of the type tested and constructed of 700-lb. units would have a strength, after three hours of fire exposure, well above the working load of 80 p.s.i. allowed for hollow masonry walls.

EFFECT OF DESIGN OF UNIT AND WALL THICKNESS

In Fig. 5 and 6 are presented the results showing the influence of design of unit on wall strength after fire exposure. Fig. 4, shows the seven types of units used and the 12 types of walls and details of mortar joints* included in the study. Both Fig. 5 and 6 show a marked tendency toward rectilinear relationship between strength of wall and strength of unit for a single type of wall. This was pointed out previously in the discussion of Diagram A of Fig. 5.

Another significant fact disclosed by Fig. 5 and 6 is the close agreement in the ratio of strength of wall to strength of unit among the different types of wall of the same thickness. With the exception of the 8 and 12-in. walls of 2-core tile units, and the 12-in. wall of 4-core tile units (Types 2, 8 and 10, Fig. 4), the values lie between 29 and 35 per cent. The comparatively low ratio of wall to unit strength of the 8-in. walls of 2-core tile units after 2 hours of fire exposure, and of the 12-in. wall of the same design of unit after 4 hours of fire exposure, indicates that this design should be bedded in mortar at the transverse webs if wall strengths equivalent to those of other designs are to be obtained. The same can be said of 12-in. walls of 4-core tile units.

The 4-in. wall, which was made of 3-core tile units (using seven different types of aggregate and mixes ranging from 1:5 to 1:14), showed a somewhat higher ratio than the 8 and 12-in. walls made from other designs. This is believed to be largely due to a more complete utilization of the available bearing area of the unit. This more complete bedding also accounts for the relatively high strengths exhibited by the 8-in. duplex and the 12-in. triplex walls (Types 7 and 12) which, it should be noted, were exposed to fire one-half hour and 3½ hours longer respectively, than the maximum periods of the other 8 and 12-in. walls.

*It should be noted in Fig. 4 that all block units were bedded in mortar applied at face shells only but that the tile units were bedded in mortar applied at the face shells and at the intermediate parallel webs.

EFFECT OF VARIOUS FACTORS OF WALL ASSEMBLY

Type of Mortar

The influence of type of mortar on wall strength is shown by the curves of Fig. 7 and 8 based on tests of 14 walls of 3-oval-core block (Type No. 3, Fig. 4) and 14 walls of 6-core tile (Type No. 4, Fig. 4). Five different types of 1:3 mortar were used as indicated in Fig. 7. One-half of the walls were tested for strength without fire exposure and one-half after exposure to fire—3 hours for block and 3½ hours for the tile.

The curves in Fig. 7 show the ratios between wall and unit strength for the different types of mortar. It was found convenient to plot the curves in this manner to conpensate for small variations in the strength of the units, which were not all made at the same time of the same lot of materials. It may be seen that all the walls both unexposed and those exposed to 3 or 3½ content hours of fire, showed a gradual in-increase in strength until the cement increased to about 70 per cent of the total volume of cementitious material used (roughly 2 volumes cement to 1 volume lime). Further increase in cement content re-

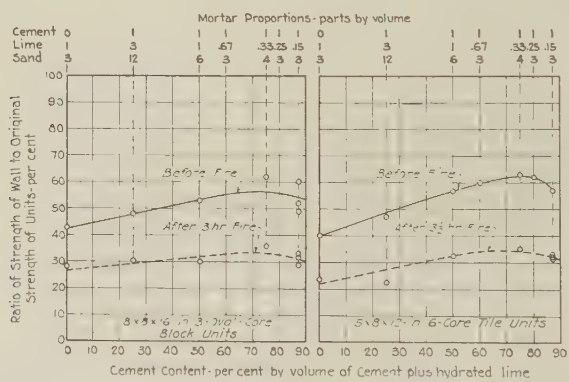

FIG. 7—EFFECT OF TYPE OF MORTAR ON RATIO OF STRENGTH OF WALL TO ORIGINAL STRENGTH OF UNITS

Block and tile units made from 1:8 mix by dry, rodded volume of calcareous sand and gravel aggregate of fineness modulus 4.1.

Walls laid with mortars consisting of 1 volume of cementitious material (portland cement plus hydrated lime) to 3 volumes of dry, rodded Elgin sand graded 0 to No. 14 sieve. These 1:3 mixes correspond approximately to 1:4 mixes prcportioned in field with damp, loose sand. Desired volumetric mortar proportions maintained by using equivalent weight proportions of dry materials considering weight of sand to be 110 lb., cement 94 lb., and lime 42 lb. per cu. ft.

Block units bedded in mortar at face shells only.

Tile units bedded in mortar at face shells and central web.

Walls exposed to fire allowed to cool to room temperature for 24 hr. before loading to failure.

Age of walls when tested: 2 months.

sulted in a falling off of strength believed to be largely due to the lack of fatness in the comparatively harsh 1:0.15:3 mortar which influenced the character of bedding.

Approximately the same wall strengths were obtained with the 1:1:6 cement-lime mortar, as with the 1:3 portland-cement mortar plus 15 per cent lime used as standard in this investigation. In this connection, it is of interest to compare the curves in Fig. 8 of strength of wall with the strength of mortar cubes and cylinders of different types of mortar.* These show that for the conditions of these tests there is no definite and direct relationship between the strength of a wall assembly and the strength of mortar cube and cylinder specimens comparable to the relationship between strength of wall and that of unit presented above. It appears that in walls of concrete masonry units other factors besides potential mortar strength come into play.

*The curve of wall strengths (in pounds per square inch of gross sectional area) for the different types of mortar is based on the same tests as the curve of ratio of wall to unit strength "before fire" in Fig. 7 for the walls of 6-core tile units.

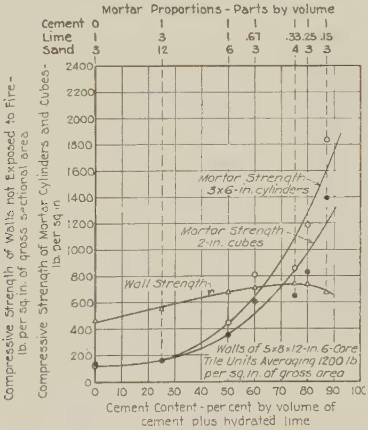

Fig. 8—Comparison of strength of walls and strength of mortar cubes and cylinders

Tile units made from 1:8 mix by dry rodded volume of calcareous sand and gravel aggregate of fineness modulus 4.1.

Walls laid with mortars consisting of 1 volume of cementious material (portland cement plus hydrated lime) to 3 volumes of dry, rodded Elgin sand graded 0 to No. 14 sieve. These 1:3 mixes correspond approximately to 1:4 mixes proportioned in field with damp, loose sand. Desired volumetric proportions maintained by using equivalent weight proportions of dry materials considering weight of sand to be 110 lb., cement 94 lb., and lime 42 lb. per cu. ft.

Tile units bedded in mortar at face shells and central web.

Age of walls when tested: 2 months.

Mortar cube and cylinder specimens excepting those of straight 1:3 lime mortar were removed from mold at 1 day; 1:3 lime mortar specimens at 5 days. After removal from mold all mortar specimens were cured in air of laboratory until tested for strength at age of 2 months.

Among these may be mentioned the inherent uniformity of dimensions and the comparatively large size of the concrete masonry units themselves. These factors in turn influence the number, thickness and uniformity of the mortar joints, thereby greatly reducing the influence of mortar strength on the strength of the wall assembly.

Attention is called to the fact that the average strength of the block and tile units represented in the curves of Fig. 7 ranged from 1200 to 1400 lb. That relatviely similar effects of type of mortar on the ratio of wall to unit strength would have been obtained with units of lower or higher strength is indicated by recent tests[4] which show that the ratio of wall to unit strength for a given type of mortar was practically constant for units ranging in strength from 300 to over 4000 lb.

Type of Mortar Joint-Workmanship

Table 3 gives the results of a group of tests to determine the improvement in wall strength that can be obtained by laying up walls of 3-oval core block with "full mortar bedding" instead of the more usual "face shell bedding" (Diagram No. 3, Fig. 4) adopted as standard construction in this investigation. The block were made from three widely different aggregates selected because of differences in their thermal properties, mineral composition, expansion, and reaction to fire exposure. The relative strength results obtained are expressed in the table for convenience as ratios of wall to unit strength which were taken from graphs based on tests of walls of each type of aggregate from units of a wide range in mix and strength.

TABLE 3—STRENGTH OF WALLS LAID UP WITH DIFFERENT TYPES OF MORTAR JOINTS

Block units made from aggregate graded from 0 to ⅜ in. and from mixes ranging from 1:5 to 1:14 by volume of cement to dry rodded volume of aggregate. Walls laid up with 1:3 portland cement mortar plus 15 per cent hydrated lime by volume of cement.

Type of Mortar Bed Construction	Application of Mortar at Horizontal Joints	Type of Aggregate	Ratio of Wall Strength after Fire Exposure Indicated to Original Strength of Unit, Per Cent		Improvement in Wall Strength with Full Mortar Bedding over Face Shell Bedding, Per Cent	
			Before Fire	After Fire	Before Fire	After Fire
Face Shell Bedding	Face Shells Only	Haydite	55	36 (3½ hr.)	..	..
		Calcareous	55	28 (3 hr.)	..	..
		Siliceous	55	22 (3 hr.)	..	..
Full Bedding	Face Shells and Transverse Webs	Haydite	70	49 (3½ hr.)	27	36
		Calcareous	70	43 (3 hr.)	27	53
		Siliceous	70	36 (3 hr.)	27	64

Comparison of the ratios of wall to unit strength show that with full mortar bed construction (through which the transverse webs as well as the face shells of the units become effective for carrying the

[4]R. E. Copeland and A. G. Timms, "Effect of Mortar Strength and Strength of Unit on the Strength of Concrete Masonry Walls," JOURNAL, Am. Concrete Inst., April, 1932, *Proceedings* V. 28, p. 551.

superimposed load) the strength of walls tested without exposure to fire was increased 27 per cent (regardless of type of aggregate) and that of walls tested after exposure to fire was increased from 36 to 64 per cent depending on the type of aggregate used.

The much greater increase in the strength of the exposed walls with full mortar bedding resulted from the more effective use of the comparatively sound interior and unexposed portions of the wall. This accounts for the substantial improvement in strength after fire exposure of the walls of highly siliceous aggregate, which as a class are most seriously affected by fire. The data show that with full mortar bed construction the strength of walls of highly siliceous aggregate compares favorably with that of walls of Haydite or calcareous aggregate laid up with the usual face shell bedding.

Effect of Plaster

The influence of plaster on wall strength after fire exposure was determined from two series of tests which will be described separately.

Tests of 8-in. Plastered Walls—This series comprised six walls of 3-oval-core blocks using two aggregates of widely different types,—calcareous sand and gravel, and Haydite. One wall of each aggregate was plastered on the fire face, one on the unexposed face, and one on both faces. The performance of these walls was compared to that of unplastered walls of otherwise identical make-up. To eliminate so far as possible the direct and indirect effects of cement content on the fire endurance period, the units selected for these tests were of the leanest mixes shown in Table 2—1:10 for Haydite and 1:14 for the calcareous aggregate. The walls were laid up with 1:0.15:3 mortar with joints as shown in diagram No. 3 of Fig. 4.

Three coats of plaster were applied to a combined thickness of not more than ½-in. The first or scratch coat consisted of "bond-crete," a wood pulp gypsum plaster applied without sand to a thickness of about 1/16-in. The second or brown coat consisted of a mixture of 1 part of hair-fibered gypsum plaster to 4 parts, by weight, of dry lake sand applied to a thickness of about ⅜-in. The third or finish coat about 1/16-in. thick consisted largely of lime putty with small additions of gypsum gaging and gypsum fibered plaster. The walls were seasoned from 45 to 60 days before plastering and from 70 to 85 days after plastering.

Since the period during which the bare and plastered walls were exposed to fire varied from 3 to 4½ hours, the effect of plaster on strength can be determined only in a general way from a comparison of the ratios in Table 4. These indicate that the strength of the walls

TABLE 4—EFFECT OF PLASTER ON STRENGTH OF 8-IN. WALLS AFTER FIRE EXPOSURE

Plastered Face	Ratio of Wall Strength, after Fire Exposure Period Indicated, to Original Strength of Units, Per Cent	
	Calcareous Sand and Gravel	Haydite
None	28 (3 hr.)	36 (3½ hr.)
Fire Face	32 (3½ hr.)	43 (3½ hr.)
Unexposed Face	29 (3½ hr.)	43 (3½ hr.)
Both Faces	39 (4 hr.)	43 (4½ hr.)

when plastered on both faces was substantially greater after 4 to 4½ hours of fire exposure than that of the bare walls after 3 to 3½ hours fire exposure. When plastered on the one face only, the strength was somewhat greater after fire exposure, but the results are not consistent for the two types of aggregate.

Tests of 4-in. Plastered Walls—This series comprised 8 walls constructed with 4 by 8 by 16-in. 3-core tile units (Diagram No. 1 Fig. 4) with 1-in. face shells and webs. The walls were built in duplicate of units made from four types of aggregates (Haydite, cinders, limestone and highly siliceous sand and gravel) selected because of differences in their thermal properties, mineral composition, and expansion, as well as for the differences which they produced in the surface texture of the units for reception of plaster.

One wall of each aggregate was plastered on both faces with a full ½-in. of gypsum plaster. This was applied in three coats of the same composition as described for the 8-in. walls. The remaining walls, one for each aggregate, were tested unplastered to bring out the influence of plaster finish. The plastered walls were, in general, exposed nearly twice as long as the unplastered walls each test being

TABLE 5—EFFECT OF PLASTER ON STRENGTH OF 4-IN. WALLS AFTER FIRE EXPOSURE

Each type of aggregate was graded from 0 to ⅜-in. with a fineness modulus of 3.50. To give units of approximately the same strength (averaging 700 p.s.i. of gross area) the following mixes by volume of cement to dry rodded volume of mixed aggregate were used: Haydite and cinders, 1:7, limestone 1:14, and siliceous sand and gravel 1:12.

The walls were constructed of 3-core tile units laid up with 1:3 portland cement mortar plus 15 per cent hydrated lime by volume of cement as detailed in Diagram No. 1, Fig. 4.

The walls were seasoned from 30 to 44 days before plastering and from 71 to 78 days after plastering in freely circulating air at the temperature and humidity of a heated building.

Plastered Face	Ratio of Wall Strength, after Fire Exposure Period Indicated to Original Strength of Unit, Per Cent			
	Haydite	Cinders	Limestone	Highly Siliceous Sand and Gravel, "A"
None	37 (1¼ hr.)	29 (1¼ hr.)	43 (1¼ hr.)	27 (1¼ hr.)
Both Faces	40 (2½ hr.)	40 (2½ hr.)	68 (2½ hr.)	41 (2 hr.)*
Percentage Increase over Unplastered Wall	8	37	58	51

*The strength of this wall would probably have been somewhat lower if exposed to fire for 2½ instead of 2 hr.

continued beyond the development of critical end-point temperatures to the next quarter or half-hour period.

Comparison of the ratios of wall strength to original strength of unit in Table 5 shows that the strength of the plastered walls after 2½ hr. of fire exposure was usually substantially greater than that of the unplastered walls after 1¼ hr. of exposure and that the improvement in strength varied with the type of aggregate. The variation in effect with type of aggregate is due to the protection afforded by the plaster which prevented the fire effects characteristic of each type. It is of particular importance to note that the walls of cinders and limestone, the two aggregates undergoing the greatest compositional changes upon fire exposure, showed a greater gain in strength when protected by plaster than the wall of Haydite. Of even greater interest and importance was the substantial improvement in the strength of the wall of highly siliceous aggregate brought about by the ½-in. of plaster.

It is of interest to report that with both the 8-in. and 4-in. walls, except for the thin finish coat of lime putty, the plaster on the fire side remained in position during the entire fire exposure and subsequent cooling period in spite of differences in the surface texture, thermal expansion and bowing of the walls. This indicates that the type of aggregate used to make the units may be varied within wide limits without impairing the protection afforded by a properly applied finish of ordinary gypsum plaster.

EFFECT OF MISCELLANEOUS FACTORS

Study of Wall Strength after Double Exposure to Fire

Tests were made to study the effect of long drying or seasoning, such as would ordinarily be encountered in a heated building on strength and fire endurance. The walls were made in pairs, one of each pair being exposed to fire and tested to failure in the usual manner after 2 months' seasoning. The other, after seasoning 1 year, was given two fire exposures separated by a cooling period of about 16 hr. during which the wall returned to room temperature. After the second fire exposure, the walls were allowed to cool and then were tested to failure in the usual manner.

One pair of walls was constructed with blocks of calcareous sand and gravel and three pairs with blocks of Haydite. Different mixes were used in the latter to study the effect of cement content. Haydite, being a material of high water absorption and water-retaining properties, was well adapted to the comparison of seasoning periods.

Because of their lower moisture content at the time of test, the walls seasoned 1 year developed critical end point temperatures somewhat earlier than the corresponding walls seasoned 60 days. The walls, however, were able to endure a second fire test 24 hr. later of the same or nearly the same exposure period and after the second test carried the same or slightly higher loads than the walls exposed once after 60 days' seasoning. These results are interesting in showing that long seasoning increased somewhat the transmission of heat, but did not change appreciably the ratios of strength of wall to original strength of unit which compared favorably with the corresponding ratios for walls exposed in the usual manner.

Influence of Coal and Coke in Cinder Aggregate on Wall Strength

The 9 walls in this group were made with the 2- and 3-core blocks and the 6-core tile (shown in diagrams No. 6, 3 and 4 respectively of Fig. 4). The walls were laid up with 1:0.15:3 mortar with joints as detailed in Fig. 4.

The cinder aggregate used as the base material from which the units were made consisted of a very good quality of well-burned soft coal cinders from a large power plant, crushed and screened to three sizes, fine (0 to No. 4), medium (No. 4 to ⅜-in.) and coarse (⅜ to ½-in.). In general, the combustible content of the various gradings of the crushed cinders averaged 12 per cent—10 per cent as coke and 2 per cent as soft coal.

The soft coal, hard coal, and coke breeze, which were added in varying amounts and proportions as described below to increase the combustible content of the base cinders, ranged from dust to lumps that would just pass a ½-in. sieve.

The strength of the walls in these 3 groups are discussed below. In general, the effect of the fire was to soften and convert the exposed face of each wall into a layer of ash from ¼ to ⅜-in. thick which served to protect the combustible matter and concrete underneath from direct contact with the fire. This protection was such that unburned coke and coal particles could frequently be found as close as ¼ in. to the exposed face after the fire exposure.

Tests of Walls of 2-Core Cinder Blocks—Two walls of units containing approximately 16 and 33 per cent respecitvely of combustible materials were obtained by adding the desired amounts of a mixture of 1 part soft coal, 1 part hard coal, and 2 parts of coke breeze to the base cinders used in making the 2-core blocks.

The blocks were made from a 1:7 mix, by volume of cement to dry rodded volume of cinders and added combustibles. The good grading,

mixing, tamping, and curing gave a dense block of excellent quality.

The strengths of the two walls after 3½ hours of fire exposure were substantially the same although the wall of cinders containing 33 per cent of combustible material showed a slightly higher ratio of strength of wall to original strength of unit (39 per cent) than the wall with 16 per cent of combustible material (34 per cent).

Tests of Walls of 6-Core Cinder Tile—Five walls were tested to determine the relative influence of combustible matter in cinders in the form of coke as distinguished from combustible matter in the form of soft coal. One wall, used as a basis of reference, consisted of tile made from the base cinders containing approximately 14 per cent of coke and 2 per cent of soft coal without the addition of either coal or coke. The other four walls consisted of tile made from the base cinders to which either soft coal or coke breeze were added to give a total combustible content of about 24 and 32 per cent. Except for combustible content, the cinder tile in the various walls were closely comparable in grading and cement content. The mix was 1:7 by volume of cement to dry rodded volume of the mixture of cinders and combustible matter. The results appear in Table 6.

TABLE 6—EFFECT OF COKE AND SOFT COAL ON STRENGTH OF 8-IN. WALLS MADE OF 6-CORE TILE UNITS

Aggregate	Total Combustible Content, Per Cent by Dry Weight of Cinders	Cement Content		Average Weight of Air-Dry Units, Lb.	Ultimate Strength, p.s.i. Gross Area		Ratio of Wall Strength to Original Stength of Units, Per Cent
		Units per Sack	Lb. per Unit		Units Unexposed to Fire	Wall after 3½ hr. Fire Exposure	
Base Cinders	16	38.5	2.44	14.9	700	185	26
Base Cinders plus 9% Coke.....	24	35.2	2.67	14.4	890	325	36
Base Cinders plus 18% Coke.....	32	36.7	2.46	13.3	750	170	22
Base Cinders plus 9% Coal......	24	36.4	2.58	13.5	740	136	18
Base Cinders plus 18% Coal.....	32	34.8	2.70	13.7	740	188	25

The ratios in the last column of Table 6 show that no definite relation exists either between the coke, coal, or total combustible content of the cinders and the strength of walls after 3½ hours of fire exposure. Although the ratios varied from 18 to 36 per cent, the average value of 25 per cent for all tests agrees very well with the ratio of 26 per cent obtained with the wall containing base cinders alone.

Tests of Walls of 3-Oval-Core Block—Two walls of 3-oval-core block were constructed, one of units made entirely of cinders and the other of units made entirely of coke breeze. The units were closely comparable as to grading and cement content (fineness modulus, cinders, coke breeze, 3.25 3.50; mix 1:7.0 and 1:7.5; cement content 4.6 and 4.4 lb. per block).

Practically identical ratios of wall strength to original strength of unit (32 per cent for cinders and 33 per cent for coke breeze) were obtained. It appears, therefore, that the cinders can be entirely replaced with combustible material in the form of coke without materially affecting the load-carrying ability of the wall either during or after 3 hours fire exposure.

Strength of Walls as Influence by Duration of Moist Curing of Units

The influence of curing was determined from three 8-in. walls of 3-oval-core block* identical as to cement content, type and grading of aggregate but differing as to the duration of the moist curing period.

Immediately after molding, the blocks were placed in a room where they were moist cured for 1, 5 and 14 days at a temperature of 70° F. and in a saturated foggy atmosphere. After 9 days in freely circulating air the blocks were laid into walls all of which were seasoned for the same period and under identical conditions before exposure to fire.

The strength of the units tested at an age of 60 days increased with duration of moist curing, the values being 1320, 1430, and 1880 p.s.i. of gross sectional area respectively for the 1, 5 and 14-day moist curing periods. This substantial increase in strength of the units was reflected in a corresponding increase in strength of the walls after 3 hr. fire exposure, which showed ultimate loads of 400, 430, and 630 p.s.i. of gross sectional area respectively. The results of these tests substantiate the general conclusion that the strength of walls after fire exposure is directly proportional to strength of units.

GENERAL COMMENTS ON WALL STABILITY DURING AND AFTER EXPOSURE TO FIRE

As might be expected, the exposure of one face of the wall to fire for periods varying from 1¼ to 9 hours and to temperatures from 1700 to 2400° F. produced changes in the physical properties and composition of the exposed materials and imposed conditions which constituted a severe test of the load-carrying ability of the walls.

One effect was to reduce the strength of the concrete and the bond between the cement paste and the aggregate by the gradual dehydra-

*Aggregate of calcareous sand and gravel 0 to ⅜-in., fineness modulus 4.1, mix 1:8. The blocks were laid in 1:3 portland cement mortar, plus 15 per cent hydrated lime with joints as detailed in Diagram No. 3 of Fig. 4.

tion of the cement, and by the expansion and changes in the physical properties of the aggregate. The highly siliceous aggregates showed the greatest thermal expansion. In general, the strength was least affected with those aggregates showing relatively low expansion.

The mortar joints were generally softened to a depth of from ½ to ¾ in. from the exposed face and usually appeared to be much more affected by fire exposure than the adjacent surfaces of the units.

Practically no fusion occurred in walls of either calcareous sand and gravel, limestone, Haydite or slag for exposures of 5 to 6½ hours during which the furnace temperatures rose to 2200° F. With exposure of 9 hours (maximum furnace temperature 2400° F.) walls of calcareous sand and gravel fused to a depth of about ⅜-in. and similar walls of Haydite fused to a depth of ¾ in.

Type of Aggregate (Graded 0 to ⅜ in.)	Expansion in $\frac{1}{1000}$ in. per ft. for Gradings Ranging from Fineness Modulus 2.0 to 4.5
Cinders, Blast-furnace slag, coke breeze common brick, fire brick, and Haydite	10 - 20
Limestone, calcareous sand and gravel	20 - 60
Highly siliceous sand and gravel	40 - 100

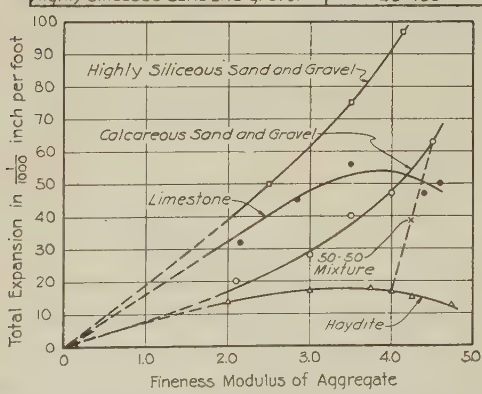

FIG. 9—EFFECT OF TYPE AND GRADING OF AGGREGATE ON THERMAL EXPANSION OF WALLS OF CONCRETE MASONRY UNITS DURING 3 TO 3½ HR. OF STANDARD FIRE EXPOSURE WITH TEMPERATURES AT THE EXPOSED FACE REACHING 1900 TO 2000° F.

Expansion

Fig. 9 and the table accompanying it give the approximate thermal expansions obtained in tests of 8-in. walls made with 3-oval core block of constant cement content (1:8 mix). The values are the average of the maximum expansions observed in each wall. It will be observed

that the expansion of the walls varied with the type of aggregate, and for those aggregates showing relatively high expansions, increased markedly as the proportion of coarse to fine particles and the denseness of the concrete increased. The wall expansions were relatively little affected by variations in the cement content of concrete made with aggregate of constant grading.

Change of Shape of Wall

The differential expansion between the exposed and unexposed faces of the walls caused them to bow or deflect toward the fire side. This is illustrated in Fig. 10 drawn to an exaggerated scale. The bowing started during the first 10 minutes of fire exposure and continued to increase to a maximum at from $\frac{1}{3}$ to $\frac{2}{3}$ the total period of exposure. The bowing then either remained constant or decreased as the wall adjusted itself to about $\frac{1}{2}$ of the maximum recorded. The maximum deflections (measured at the center of the unexposed face) varied from $\frac{1}{8}$ to $\frac{5}{8}$-in.

Accompanying this bowing of the wall, a separation developed between the curved bearing edges at the top of the wall and the straight bearing edge of the test frame. The maximum separation (measured at the vertical center line on the unexposed face) varied from about $\frac{1}{64}$ to $\frac{1}{8}$ in. While no corresponding separation developed at the bottom of the wall, it was quite evident that the loading beam was bent under the thrust of the four hydraulic jacks to conform to the curved shape of the lower bearing edge.

Change in Character of Loading Due to Bowing of Wall

It will be apparent from the diagrams in Fig. 10 that the uniformly distributed axial loading before fire exposure changed to a more or less concentrated and eccentric loading, diagrams (b) and (c), with changes in shape and dimensions of wall during fire exposure. It is obvious, therefore, that for a greater part of the test period the hot material on the exposed face of the wall had to resist compressive stresses substantially greater than would normally result from a uniformly distributed working load of 80 p.s.i. of gross bearing area. The fact that all but one of the 165 walls exposed to fire carried this superimposed working load under adverse conditions during the entire fire exposure and cooling period adds to the significance of the data that have been presented regarding the general load-carrying ability of the walls as determined from the strength tests after exposure to fire.

With the readjustments that took place during the cooling period the bowing of the wall toward the exposed face decreased and eventually

reversed in curvature toward the unexposed face. This later bowing was permanent and maximum deflections from $\frac{1}{16}$ to $\frac{3}{8}$-in. were recorded toward the unexposed face when the walls had cooled to room temperature. This resulted in shifting of a considerable proportion of the load applied by the hydraulic jacks to the unexposed portion of the wall. During the subsequent application of load, therefore, the walls were again subjected to a more or less eccentric loading concentrated mainly on the unexposed portion of the wall.

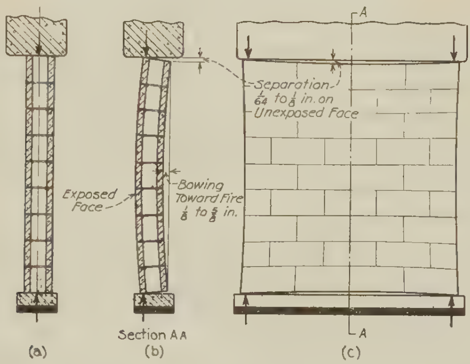

FIG. 10—DIAGRAM ILLUSTRATING CHANGES IN LOADING AND IN SHAPE OF WALL

The diagrams show to an exaggerated scale how the uniformly distributed axial loading before fire exposure was changed to a more or less concentrated and eccentric loading with changes in shape and dimensions of wall during fire exposure.

Cracking

In general two or three vertical cracks were formed on the unexposed face of the wall during the first ½ hour of fire exposure. These followed the vertical mortar joints and the intermediate units. Usually one of the cracks was near the vertical center line and the other two about midway between this central crack and the edges of the wall. Horizontal cracks always followed the mortar joints but their location was not as regular as that of the vertical cracks. Diagonal zig zag cracks following both horizontal and vertical joints were sometimes formed at the corners of the wall.

The maximum width of the cracks varied from about $\frac{1}{64}$ to $\frac{1}{8}$-in. for the vertical and from very fine cracks to about $\frac{1}{16}$ in. for the hori-

zontal. During cooling, the width of the cracks decreased and seldom exceeded $\frac{1}{32}$ in. after the wall had cooled to room temperature.

While no cracks were visible on the exposed face during fire exposure they became apparent after several hours of cooling. The vertical cracks were, in general, found to coincide with the major cracks on the unexposed face. Other minor cracks were formed in the vertical and horizontal mortar joints. In general, the whole units were divided into half units by the cracks passing through the vertical mortar joints in the courses above and below.

Spalling

In no case was there any spalling of the concrete either during or after fire exposure. An occasional local disruption and pitting of aggregate particles at the exposed face of the walls was found in the case of Haydite and air-cooled blast-furnace slag, but these local effects did not appear to affect the load-carrying ability or fire endurance of the walls.

Modulus of Elasticity of Walls

The stress-strain curves showed fairly consistent relations between the applied load and the resulting deformation both for the walls tested for ultimate strength after exposure to fire and those tested without exposure. In general the curves were practically straight lines for loads up to about 90 per cent of the ultimate strengths obtained. For the walls exposed to fire, two stress-strain curves were available, one based on the application of a load equivalent to 3 times the working load of 80 p.s.i. of gross area before fire exposure and the other based on the test for ultimate strength after fire exposure. These curves afforded a means of comparing the modulus of elasticity of the same wall before and after fire.

The modulus of elasticity after exposure was always less than that before exposure. Based on the slope of the straight-line portion of the stress-strain curve the values before exposure ranged from about 200,000 to 750,000 p.s.i. and after exposure from about 100,000 to 300,000 p.s.i. of gross area. The moduli after exposure averaged about 55 per cent of that before fire exposure for the 4-in. walls, 40 per cent for 8-in. walls and 30 per cent for the 12-in. walls. The percentages for all walls tested ranged from a minimum of 25 per cent to a maximum of 70 per cent.

GENERAL COMPARISON OF STRENGTH OF WALLS AFTER FIRE EXPOSURE TO ORIGINAL STRENGTH OF UNIT

Fig. 11 summarizes the results of the strength tests for the 165 walls which were exposed to fire. The brackets at the top of the diagram

indicate the range in the fire exposure periods obtained with each thickness of wall.

Approximately 11 per cent of the 165 walls had strengths of 40 per cent or more of the strength of the units, 58 per cent had strengths 30 per cent or more and 92 per cent had strengths 20 per cent or more. Only about 7 per cent of the walls had strengths between 15 and 20 per cent of the strength of the units.

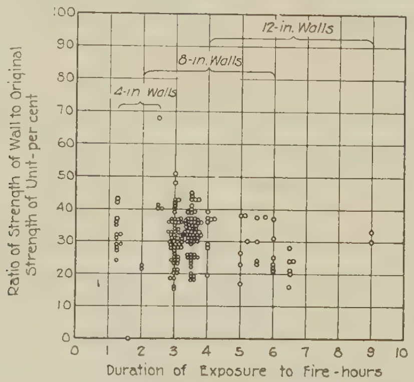

FIG. 11—COMPARISON OF RATIO OF STRENGTH OF WALL TO ORIGINAL STRENGTH OF UNITS AFTER ENDURANCE TESTS

The plotted points represent a total of 165 tests which include 4, 8 and 12-in. walls of various types. In order to show a point for each test on the diagram, it has been necessary to plot values falling at the same point to the right and left of the actual period of fire exposure to avoid confusion.

Only one of the 165 walls failed during fire exposure under the working load of 80 p.s.i. of gross area. This wall was constructed of units made from coarsely-graded highly siliceous aggregate (fineness modulus 4.25) and a very lean mix (1:14) as shown by Table 2. It represented a critical combination which was unable to resist the high concentration of load on the exposed face due to expansion. The wall buckled after 93 minutes of exposure to fire. This indicates that when highly siliceous aggregates are used, coarse gradings with excessively lean mixes should be avoided. That satisfactory performance can be obtained with this type and grading of aggregate in a richer mix, is demonstrated by the data in Table 2 where a wall constructed of units of 1:7.5 mix showed, after 3 hours of fire exposure, a ratio of strength of wall to original strength of unit of 26 per cent.

EVALUATION OF STRENGTH OF LARGE WALLS AFTER FIRE EXPOSURE

The usefulness of this investigation on walls 5½ ft. wide and 6 ft. high has been greatly extended by the recent strength tests at the

University of Illinois on large concrete masonry walls 6 ft. wide and 9½ ft. high not exposed to fire, and by the tests on large walls conducted from time to time since 1921 at the Underwriters' Laboratories, Chicago. The latter tests cover the performance of concrete masonry walls 10 ft. wide and 11 ft. high when subjected to standard fire endurance, water stream, and excess load tests. Correlation of the results obtained from various phases of all three investigations enables an estimate to be made of the probable strength after fire exposure of large walls comparable in make-up to the 5½ by 6-ft. walls used in this investigation.

University of Illinois Tests

The tests conducted at the University of Illinois have been described by Richart, Woodworth, and Moorman[5] and are also fully discussed in the recent paper by Professor Richart.[6] Among other improtant results they showed a linear relationship between strength of wall and unit both for large walls 6 ft. wide and 9½ ft. high and for wallettes 32 in. wide and 48 in. high. They also showed that the ratio between the compressive strength of large walls and comparable wallettes was fairly constant with an average value of 0.91. These results demonstrate the feasibility of judging the strength of large walls from that of small walls of similar make-up.

Underwriters' Laboratories Tests

Tests carried out at the Underwriters' laboratories on hollow concrete masonry walls of various designs of units, type of aggregate, and wall thickness, have given interesting data on the strength of walls after exposure to fire from 1¾ to 7⅓ hrs. No published report has been made on all of these tests. The following information concerning the test procedure and results is given with the permission of the Underwriters' Laboratories.

Generally, in the Underwriters' tests, two identical walls 10 ft. wide by 11 ft. high are constructed of units laid up in 1:3 portland cement mortar containing 15 per cent of hydrated lime, usually with full mortar joints. One is subjected to the Standard Fire Endurance Test while carrying a superimposed working load of 80 p.s.i. of gross bearing area until the critical end-point temperatures develop on the unexposed surface. This wall is then withdrawn from the furnace and kept under load until it has cooled to room temperature. The load is then removed and the wall dismantled for a detailed examination of the individual units. The other wall is subjected to Standard Fire

[5]Tests of the Stability of Concrete Masonry Walls, by F. E. Richart, P. M. Woodworth, and R. B. B. Moorman, Proc. Am. Soc. Testing Mat., V. 31, Part II, pp. 661-680, 1931.

[6]The Structural Performance of Concrete Masonry Walls, by F. E. Richart; JOURNAL Am. Concrete Inst., Feb., 1932, *Proceedings*, V. 28, p. 363.

Exposure for one hour and then immediately exposed to the impact, cooling, eroding, and wetting effects of a standard water stream applied to the incandescent face. After cooling for approximately 24 hours, it is subjected to the excess load test during which it is loaded to twice the working load of 80 p.s.i. Usually, this wall is also dismantled for examination of the units.

In some of the tests, however, it was found desirable to subject a single wall to all of the tests (fire endurance, water stream and double load test) ordinarily given two walls. Under this plan of testing, a 12-in. wall of units made from four types of aggregates withstood the water stream and double load tests after exposure to fire for 7⅓ hours and to an ultimate temperature of 2225° F.

In recent tests at the Underwriters' Laboratories, further information regarding the stability of 8-in. walls was obtained. A brief description of these tests follows:

(1) An 8-in. wall of 2-core block units (of the type illustrated in diagram No. 6 of Fig. 4) made from cinders containing additions of raw hard coal, soft coal and coke in the proportion of 1:1:2 by weight and totalling 20, 35, and 45 per cent showed a ratio of wall strength to original strength of unit of 25 per cent after 3 hours fire exposure and a cooling period of 20 hours. Failure occurred by crushing in the upper third section of wall consisting of units made of cinders having a total combustible content of 45 per cent. The units had an average strength of 1000 p.s.i. gross area and were bedded in mortar applied at face shells only as in diagram No. 6 of Fig. 4.

(2) An 8-in. wall of 3-oval core block units (of the type illustrated in diagram No. 3 of Fig. 4) made from Haydite aggregate obtained from six sources was exposed to fire endurance test for 3 hours and then to the water stream test. After cooling for 20 hours, it showed a ratio of wall strength to original strength of unit of 37 per cent. The units had an average strength of 895 p.s.i. gross area and were laid up with full mortar bedding.

(3) Two 8-in. walls of a 3-oval core unit of special design having relatively thin face shells (1¼ in. average) and webs (1 in. average) and made from Haydite aggregate were constructed. The units had an average strength of 780 p.s.i. of gross area and were laid up with full mortar bedding.

One wall was tested for ultimate strength in the hot condition immediately after exposure to fire for 2⅔ hours and gave a ratio of wall strength to original strength of unit of 35 per cent. The other wall was tested for ultimate strength in the damp condition 20 hours after it had been exposed to fire (one hour) and the water stream test and showed a ratio of wall to unit strength of 39 per cent.

Probable Strength of Large Walls after Fire Exposure

In an effort to obtain a direct basis for comparison between the 5½ by 6-ft. walls used in this investigation and the 6 by 9½-ft. and 10 by 11-ft. walls used in the University of Illinois and Underwriters' Laboratories tests, a careful study has been made of the three sets of strength data. Besides the difference in size and ratio of height to width, there are other factors which influence the comparison. Among

these, the most important are the type of bedding, type of mortar, the workmanship used in laying up the walls, and the moisture content and age of the completed walls and individual units. On the basis of such comparisons as are offered by the data, it appears that the over-all effects of these various factors are such that the strength of the larger walls after fire exposure can safely be estimated at 75 per cent of the strength of corresponding small walls after a similar period of fire exposure.

Applying this percentage to the results of the tests on 8-in. walls constructed of 3-oval core block made from different aggregates and laid up with portland cement, lime mortars ranging from 1:1:6 to 1:.0.15:3 and with either face shell or full mortar bedding, the following ratios of strength of large walls to strength of units, expressed as a percentage, are obtained:

Type of Aggregate	Face Shell Bedding	Full Bedding
Haydite, Limestone	27	37
Calcareous sand and gravel, cinders, slag, coke breeze, crushed common and fire brick	21	32
Highly siliceous sand and gravel	17	27

The above values bring out clearly the marked advantage of full bedding over face shell bedding on strength of wall after exposure to fire. It is evident that much higher wall strengths after fire exposure would have been obtained in this investigation if the various types of walls illustrated in Fig. 4 had, where possible, been constructed with full mortar bedding. This conclusion is confirmed by the results presented in Table 3.

Readers are referred to the JOURNAL *for March, 1933, for discussion which may develop. Such discussion should reach the Secretary by January 1, 1933.*

Design of Continuous Arches on Elastic Piers

BY A. P. HJORT*

INTRODUCTION

The following shows an arrangement for figuring continuous arches on elastic piers by combining the single arch design and the slope deflection method for ordinary frames.

The slope deflection method has been applied to continuous arches long ago. About 1920, Professor G. A. Maney, the originator of the method in the United States, in a discussion about reinforced concrete ships, stated a solution of transverse frames with curved members, which in principle is similar to that for continuous arches. And in Europe Professor A. Ostenfeld, Copenhagen, in his book "Die Déformationsmethode," gives a complete solution of continuous arches on elastic piers.

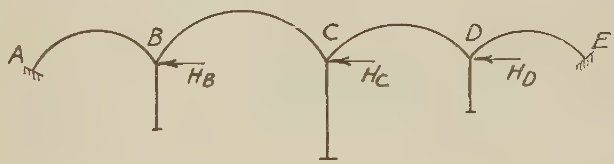

Fig. 1.

This article intends, however, to show in a little different way how a designer familiar with the ordinary slope deflection method for straight frames and the design of a single arch through simple statics can combine these two methods to a solution of continuous arches on elastic piers.

COMPUTATION

The arch shown in Fig. 1 is fixed at A and E, fixed or hinged at bottom of columns, and the elevations of A, B ... are all the same, but the arches can all be unsymmetrical. The computation is made in the following steps.

Step 1

The first step is to treat the arches A-B, B-C ... separately as single fixed arches and figure the following values:

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(*a*) The moment at A and B for the load in question.
(*b*) Same moments for an angular turn Θ of A.
(*c*) Same moments for an angular turn Θ of B.
(*d*) Same moments for a horizontal deflection d of A.
(*e*) Same moments for a horizontal deflection d of B.
(*f*) The horizontal thrust produced by an angular turn Θ of A.
(*g*) Same for a turn Θ of B.
(*h*) Same for a horizontal deflection d of A.
(*i*) Same for a horizontal deflection d of B.

The similar values are to be figured for the rest of the arches B-C, C-D and D-E.

Step 2

It is now possible to find the deflection of the whole structure for a horizontal load H_B in B, H_C in C and H_D in D. This deflection will consist of a horizontal deflection d_B, d_C and d_D and an angular turn Θ_B, Θ_C and Θ_D of the corresponding points B, C and D.

After the deflection, when the structure is in equilibrium, each arch span and each column will have produced a certain horizontal reaction to the movement: ΔH_1, ΔH_2 ... for the arch span 1, 2 ... and C_B, C_C ... for the columns.

One equation for equilibrium is therefore:

$$\Delta H_1 + \Delta H_2 + C_B = -H_B \quad (1)$$

and similar for C and D:

$$-\Delta H_2 + \Delta H_3 + C_C = -H_C \quad (2)$$

$$-\Delta H_3 + \Delta H_4 + C_D = -H_D \quad (3)$$

From Step 1, (*f*) to (*i*) we find directly:

$$\Delta H_1 = f(d_B, \Theta_B)$$
$$\Delta H_2 = f(d_B, d_C, \Theta_B, \Theta_C)$$
$$\Delta H_3 = f(d_C, d_D, \Theta_C, \Theta_D)$$
$$\Delta H_4 = f(d_D, \Theta_D)$$

Furthermore we find from simple statics the column reactions C_B, C_C and C_D expressed by (d_B, Θ_B), (d_C, Θ_C) and (d_D, Θ_D).

Substituting in (1), (2) and (3) we have three equations and six unknowns.

Three more equations can be derived from the second equation for equilibrium, which says that the sum of the moments around any of the points B, C and D shall equal zero or:

$$M'_B + M''_B + M'''_B = 0 \quad (4)$$
$$M'_C + M''_C + M'''_C = 0 \quad (5)$$
$$M'_D + M''_D + M'''_D = 0 \quad (6)$$

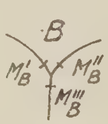

Fig. 2

Each of these moments is caused by the horizontal deflection d_B, d_C and d_D and the angular turns Θ_B, Θ_C and Θ_D.

From Step 1 (*b*) to (*e*) we find:

$M'_B = F(d_B, \Theta_B)$
$M''_B = F(d_B, d_C, \Theta_B, \Theta_C)$
$M'_C = F(d_B, d_C, \Theta_B, \Theta_C)$
and similar for M''_C, M'_D and M''_D.
From simple statics we find M'''_B, M'''_C and M'''_D.

By substituting these values in (4), (5) and (6) we have three more equations, and d_B, d_C, d_D and Θ_B, Θ_C, Θ_D can be found. This set of equations has only to be solved once for all to give the influence values of the deflections.

Step 3

If an inelastic string is applied from A to B, from B to C etc. the system can be figured as a frame by the slope deflection method for frames.

The coefficients in the equation's left side are derived from Step 1 (*b*) and (*c*) with a contribution from the columns as usual. The equation's right side are the moments in Step 1 (*a*).

The turns of B, C and D can now be found by solving the three equations, and the tension in the string A-B, B-C . . . can be figured as the sum of the horizontal thrusts corresponding to condition Step 1 (*a*) + (*f*) + (*g*) plus the thrusts from adjacent columns, which can be determined by simple statics.

The tensions in the strings being H_1, H_2, H_3 and H_4 for span 1 to 4, we have:

$$H_2 - H_1 = H_B, \qquad H_3 - H_2 = H_C \quad \text{and} \quad H_4 - H_3 = H_D$$

and the deflections caused by the string, when cut, can therefore be found directly from Step 2.

All deflections can now be added up and transformed into moments, and the stresses in any point of the structure can be determined.

ILLUSTRATIVE EXAMPLE

The arch shown in Fig. 3 is fixed at A and D and at the bottom of of the columns. The single arches are all parabolas. The thickness of the rib is given by the formulas:

$t = 10$ in. $+ 0.1377\,x$ for A $- B$, $\qquad t = 14$ in. $+ 0.1116\,x$ for $B - C$
$t = 12$ in. $+ 0.1230\,x$ for $C - D$

where x is in feet and measured from the centerline. All other dimensions are given in Fig. 3. Clockwise turns of B and C are considered

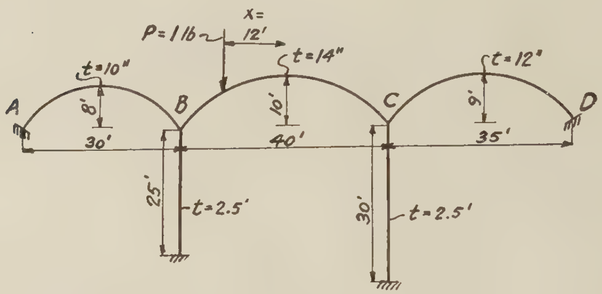

Fig. 3.

plus, and translations to the left of same points are plus. In the following computation feet and pounds are used. Modulus of elasticity, E, is considered to equal 1.

After the arches have been treated in accordance with Step 1, we can now build up the formulas in Step 2 as follows:

$$
\begin{aligned}
\Delta H_1 &= +0.004106\theta_B && && -\ 0.000662d_B && \\
\Delta H_2 &= +0.006177\theta_B && -\ 0.006177\theta_C && -\ 0.000803d_B && +\ 0.000803d_C \\
C_B &= -0.012500\theta_B && && -\ 0.001000d_B && \\
\hline
(1)\quad & -0.002217\theta_B && -\ 0.006177\theta_C && -\ 0.002465d_B && +\ 0.000803d_C = -H_B
\end{aligned}
$$

$$
\begin{aligned}
-\Delta H_2 &= -0.006177\theta_B && +\ 0.006177\theta_C && +\ 0.000803d_B && -\ 0.000803d_C \\
\Delta H_3 &= && +\ 0.005190\theta_C && && -\ 0.000746d_C \\
C_C &= && -\ 0.008681\theta_C && && -\ 0.000579d_C \\
\hline
(2)\quad & -0.006177\theta_B && +\ 0.002686_C && +\ 0.000803d_B && -\ 0.002128d_C = -H_C
\end{aligned}
$$

$$
\begin{aligned}
M'_B &= -0.0408\theta_B && && +\ 0.00411d_B && \\
M''_B &= -0.0766\theta_B && +\ 0.0293\theta_C && +\ 0.00617d_B && -\ 0.00617d_C \\
M'''_B &= -0.2083\theta_B && && -\ 0.01250d_B && \\
\hline
(3)\quad & -0.3257\theta_B && +\ 0.0293\theta_C && -\ 0.00222d_B && -\ 0.00617d_C = 0
\end{aligned}
$$

$$
\begin{aligned}
M'_C &= +0.0293\theta_B && -\ 0.0766\theta_C && -\ 0.00617d_B && +\ 0.00617d_C \\
M''_C &= && -\ 0.0579\theta_C && && -\ 0.00519d_C \\
M'''_C &= && -\ 0.1736\theta_C && && -\ 0.00868d_C \\
\hline
(4)\quad & +0.0293\theta_B && -\ 0.3081\theta_C && -\ 0.00617d_B && -\ 0.00770d_C = 0
\end{aligned}
$$

Solving equations (1), (2), (3) and (4) we find:

$$
\left.
\begin{aligned}
\theta_B &= -8.83H_B - 14.67H_C \\
\theta_C &= -16.29H_B - 21.07H_C \\
d_B &= +520H_B + 256H_C \\
d_C &= +201H_B + 582H_C
\end{aligned}
\right\} \text{Step 2}
$$

In Step 3 the moments at B and C must be figured for the arch B-C fixed at both ends. These moments are:

$M_{BC} = +4.801$ and $M_{CB} = +0.909$ (trying to turn B and C clockwise)
Now the equations in Step 3 will be—see equations (3) and (4) in Step 2:

$$-0.3257\Theta_B + 0.0293\Theta_C = -4.801$$
$$+0.0293\Theta_B - 0.3081\Theta_C = -0.909$$

Solving gives: $$\left.\begin{aligned}\Theta_B &= +15.13\\ \Theta_C &= +\ 4.39\end{aligned}\right\}\text{Step 3}$$

The tension in the string B-C, for B and C fixed will be the same as the crown thrust or 0.224.

Therefore $H_B = +0.224$ and $H_C = -0.224$
Substituting these values in Step 2 we find:

$$\left.\begin{aligned}\Theta_B &= +1.31\\ \Theta_C &= +1.08\\ d_B &= +57.8\\ d_C &= -84.7\end{aligned}\right\}\text{Step 2}$$

Adding these values to Θ_B and Θ_C in Step No. 3 we find the final deflections of the structure

$$\begin{aligned}\Theta_B &= +1.31 + 15.13 = +16.44\\ \Theta_C &= +1.08 + \ 4.39 = +\ 5.47\\ d_B &= +57.8\\ d_C &= -84.7\end{aligned}$$

and it is now an easy matter to find moments and thrusts in any point of the structure.

Readers are referred to the JOURNAL *for March, 1933, for discussion which may develop. Such discussion should reach the Secretary by January 1, 1933.*

Discussion of a paper by Thomas T. Towles:

"Advantages in the Use of High Strength Concretes"*

Ivan Rosov† (by letter): Mr. Towles has brought timely attention to advantages in the use of high-strength concretes. It must be admitted that too much timidity is displayed in this section of engineering. We have few examples of the employment of high-strength cement with a corresponding increase in working stresses for the structure. Europe advanced much farther; it is enough to mention the Albert Louppe bridge with its record spans of 567 ft. and the concrete developing the crushing strength of 5675 p.s.i.[1]

The reasons for this backwardness are probably in the complexity of the problem which far exceeds that of the qualities of the cement alone. In a common concrete structure, the low working stresses are a safeguard against possible defects in the choice of materials and in making, placing and curing concrete. Certainly, it involves a great waste. In the highly stressed concrete structures this waste should be eliminated and safety secured by the most careful examination of all conditions, most important of which are: (1) Grading and quality of aggregates; (2) Uniformity of concrete after placement; (3) Influence of the shrinkage of concrete on the stresses in reinforcement.

Discontinuity in any structural material produces local stresses which may counterbalance its most excellent qualities. If cement of high strength is employed, voids in concrete must be eliminated, that is, the density increased as much as possible. This is mostly a matter of aggregates.

The Louppe bridge above mentioned supplies the best example. Its concrete was made of high-quality-cement and broken quarzite aggregate, considered excellent from the point of view of grading and strength of particles. However, the concrete failed to show the designed resistance of 4250 p.s.i. Study led to the conclusion that the mixture had insufficient "lubrication" because of sharpness of fine particles. The addition to the sand of 50 per cent of cliff sand, poor in quality by itself but well rounded and extra fine, of about uniform size of $\frac{3}{64}$ in. (1 mm.) so improved the mixture that the crushing strength

*Journal, Am. Concrete Inst. May 1932; *Proceedings*, V. 28 p 607.

†3021 Briggs Ave., Apt. 1, Bronx, New York.

[1]*Genie Civil*, 1930.

exceeded that designed by 34 per cent. The concrete became much denser, indicating the great role of the added "lubrication" in the mobility of particles.

Just this problem of lubricating the elements of concrete is not cleared up sufficiently. There are many studies how to make a "good" mixture, but only a few on how to get an "excellent" one. Professor Bolomey, well known by his research work, supposes[2] that the plasticity largely depends on the content of sand less than 0.1 mm. (0.004 in.). On his opinion particles of this size possess the ability to hold a great deal of water and to form the kind of emulsion which lets larger grains "swim" or be suspended. If the emulsion has not sufficient specific gravity, aggregates fall down and concrete works badly. Professor Graf includes in this category all elements passing 0.2 mm. sieve and asserts[3] that the content of "extra fines" should not be less than 25 per cent of the dry mortar. In the Louppe bridge a much larger size proved best for lubricating. So the study of aggregate should be carried further and the designer of a high-strength concrete cannot omit this point.

The uniformity of concrete in common structures is by no means of a high degree. Even experimental arches made with particular care developed great discrepancies in the crushing strength of different samples. Engineers have worked out methods of control in proportioning, mixing and curing and the lack of uniformity is usually attributed to the manner of the placement.

The process of vibration recently introduced deals with this defect. Tests at the Arlington Experimental Farm indicated that the vibrated concrete acquires all qualities of heavy liquid. The vibration of forms, however, does not spread far into a concrete mass and can be applied successfully only to thin members.

A new French method, "pervibration"[4], promises to overcome this difficulty finally. Hydraulic or electric hammers are carried in steel floats which are placed on the bottom of a structure (inside forms). While concrete is being placed, vibration of the floats renders it fluid so that the floats "swim" up, spreading their action in newly placed concrete. This system works automatically, because floats cannot "swim" up unless the surrounding concrete is completely liquified. The tests show that the "pervibrated" concrete possesses great uniformity and density. The samples taken at different parts of one structure from the concrete in place showed variations in the crushing strength of 6 per cent only (from 4353 to 4606 p.s.i.). In the other[5]

[2]"Bestimmung druckfestigkeit von Mörtel und Beton," *Schweizerische Bauzeitung*, 1926.
[3]Auslau des Mörtels und des Beton," by Graf.
[4]"Béton vibré at Pervibré," *Revue Industrielle*, 1931
[5]"La Pervibration," *Genie Civil*, 1931.

case the density of the concrete was measured as the requirement of aggregate per 1 cu. yd. of concrete. This was as follows:

for common concrete —1.2 cu. yd.
for vibrated concrete —1.3 to 1.4 cu. yd.
for pervibrated concrete—1.45 cu. yd.

Combined vibration and artificial pressure used by Mr. Freyssinet developed the crushing resistance for concrete "in mass" as high as 8500 p.s.i. Samples were taken from the concrete in the structure. It is much more than Mr. Towles considers practically possible. In daily manufacturing of precast concrete the strength of 14000 p.s.i. is attained.[6]

The high stressed concrete permits the use of the compression steel in a much more economical way. Within the concrete stressed up to 650 p.s.i. the steel may carry only 15 x 650 = 9750 p.s.i. If concrete shows the working stress of 1850 p.s.i. and n equals 10, the stress in the steel may be 10 x 18750 = 18750 p.s.i., that is up to the full working strength of the metal.

The shrinkage of concrete usually precludes the use of this advantage. The customary shortening of the concrete in hardening is about 0.0004, so the reinforcement receives the initial compression of 0.0004 x 30,000,000 = 12,000 p.s.i. and only the balance of 6750 p.s.i. is available for loading stresses.

The method introduced recently in Europe and called "autofrettage"[7] came to meet this situation. Just after placing the concrete, the steel is submitted to the action of special steel device which imposes on the reinforcement an initial tensile stress pressing the concrete at the same time. When the concrete is setting and shrinking, the steel is pressed and its tension decreases. At the end of the shrinkage process, the steel has neither tension nor compression and can be stressed by outside loading from zero to the full value of 18000 p.s.i. Using an arch with 15 per cent of reinforcement, and high-strength concrete of 1850 p.s.i., it is possible to get the working stress of 0.15 x 18000 + 0.85 x 1850 = 4273 p.s.i.

All this indicates that difficulties in the use of high strength cements, complicated as they are, can be overcome and designers may develop cheap and safe structures exceeding by far the present records. It is possible to reach much greater results than Mr. Towles assumes. Freyssinet, the famous French concrete engineer, asserts[8] that, by means of the "frettage" or "hooping" and the use of the high strength cement, it is possible to raise the crushing resistance of the concrete to

[6] "A Study of the Views of Freyssinet" *Structural Engineer*, 1931.
[7] "Pont des Ibis," *Genie Civil* 1931.
[8] *Memoires de la Societe des Ingenieurs Civils de France*, 1930.

more than 40,000 p.s.i. With such a concrete the arches of 6000 ft. could be built with the limit imposed by the centering, not by the concrete arch itself. Mr. Freyssinet produced an elaborately designed scheme, for a bridge of 3270 ft., the rival of the Washington Bridge —with a factor of safety of from 4 to 5.

It is not too enthusiastic to say that Mr. Freyssinet's assumptions are well based. The crushing resistance of material depends on the difference between longitudinal and lateral pressures. Common spirally reinforced columns and their acknowledged great resistance serve as a conspicuous example. However, the spiral steel is only an imperfect method to attain the desired results. Messrs. Caquot & Brice conducted the tests which showed that any form of the reinforcement—so long as it prevents the bulging of the concrete in all directions, produces the increase in the compression resistance of the concrete. Mr. Freyssinet found that the greatest efficiency is obtained, if the transverse reinforcement is distributed in two ways normal to the acting pressure, and connected by semicircular wires. This arrangement permits the use of "frettage," much wider than for the columns only, for any pressed section of beams, slab, arches, etc.

The use of especially strong steel for transverse hooping reinforcement permits the development of very high internal compression normal to that due to outside loading and to increase in this way the resistance of the concrete to the acting stresses. This is a great advantage of reinforced concrete compared to steel, which opens for high strength cements particularly wide perspectives.

Discussion of a paper by I. Oesterblom:

"Bending and Torsion in Horizontally Curved Beams"*

Norman M. Stineman† (by letter): It seems that these formulas might be applied, perhaps with some modifications, to the analysis of horizontal wind stresses in the ribs of reinforced concrete arch bridges of long span. The rib, of course, does not have a uniform cross-section, whereas Mr. Oesterblom's formulas are based on a uniform cross-section throughout the length of the beam. As a consequence, some modification would be necessary to take account of that condition; but the problem is similar, except that it is turned through 90 degrees.

The need for a more definite analysis of horizontal wind stresses in arch ribs becomes more urgent as engineers are designing constantly increasing span lengths in reinforced concrete arch bridges. The lack of published data on this subject is well illustrated from the fact that a recent text on reinforced concrete design and construction, published in several volumes and devoting 200 pages to reinforced concrete arch bridges, contains just four lines on the subject of lateral wind pressure.

K. Hajnal-Konyi‡ (by letter): Since the author says in his introduction that scant attention has heretofore been given to horizontally curved beams, and that at most they have been treated as a statically determinate system, I wish to call attention to the fact that in German a voluminous literature is available on circular or curved beams loaded perpendicular to their planes, in which the problem has been solved far beyond the case studied by Mr. Oesterblom.

Probably the first study was that by Grashof[1] who studied the elasticity of helical springs and of a section of a ring loaded by two perpendicular forces. In the following decade among others, to mention only the most well-known names, Koenen, Müller-Breslau, Federhofer, Marcus and Andrée, studied this field. It is unnecessary for me to go further into these studies because a review of the complete literature up to 1921 is found in the fundamental book of Unold[2] which is solely devoted to the circular beam. Further reference is to Düsterbehn's

*Journal, Am. Concrete Inst., May 1932; *Proceedings*, V. 28, p. 597.
†Editor of *Concrete*, Chicago, Ill.
‡Darmstadt, Germany.
[1]F. Grashof: "Theorie der Elastizität und Festigkeit," 2nd Ed., Berlin, 1878, p. 294.
[2]G. Unold: "Der Kreisträger, Forschungsarbeiten auf dem Gebiete des Ingenieurwesens," Heft 255, Berlin 1922, (79 pages).

treatise[3] in which influence lines for different bending and torsional moments and reactions are developed for half circular beams on three or five symmetrically arranged supports and for full circles on three and four symmetrical supports.

Unold has solved Federhofer's differential equation for the elastic curve for a number of general cases, of which the following will be mentioned: Circular beams with complete rigid support and arbitrary angles for the concentrated loads and for uniformly distributed load, closed circular beams on three or more supports for concentrated loads, partially distributed load and twisting moments, always using a circle or circular cross-section.

Unold has also given the solution of numerous special cases of the circular beam having I-cross section.

While all the above given studies were given to steel construction, Hessler[4] has studied the rectangular reinforced concrete section and in 1927 published formulas which are very similar to those arrived at by Mr. Oesterblom. In fact, Hessler's formulas for M_B and M_T and Oesterblom's equations 13 and 14 become identical when the necessary changes due to the differences in notation have been made. Hessler in his first paper has not only treated the case of a uniformly distributed load but also the case of two symmetrical concentrated loads and one concentrated load placed in the center. His second paper contains interesting special cases of the half and full circular beam on several uniformly spaced supports with uniformly distributed load.

A general and very excellent solution of the stress distribution in horizontal beams loaded and supported perpendicularly has been published by Worch.[5] The circular beam is a special case of such beams. Worch gives the deformation energy for the values in the statical determinate system on three supports and makes use of these values in solving for the excess supports. The method is very suitable for the determination of influence lines.

This problem is the connecting link to the other field of plane systems, namely, to the computation of frames which are loaded perpendicular to their plane. It is not necessary to go further into the rather voluminous literature which deals with this problem.

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[3]Dusterbehn: "Einflusslinien Ringförmiger Träger," *Der Eisenbau*, 1920, No. 4, p. 73.

[4]St. Hessler: "Der Nach Einem Kreisbogen Gekrummte, Beiderseits Eingespannte Eisenbetonträger mit Rechteckigem Querschnitt," *Beton und Eisen*, 1927, Heft 24, p. 429.

St. Hessler: "Der Kontinuierliche Halbkreisförmig Gebogene und Gleichmässig Belastete Eisenbetonträger mit Rechteckigem Querschnitt auf 3 und 4 Gleich Weit Entfernten Stützen," *Beton und Eisen* 1930, Heft 8, p. 149.

[5] G.Worch: "Beitrage zur Ermittlung der Formänderungen Ebener Stabzuge mit Räumlicher Stutzung Nebst Anwendung auf die Berechnungstatisch Unbestimmter Systeme," *Beton und Eisen* 1930, Heft 9, 10 and 11 (p. 167, 183 and 200).

The practical value of Mr. Oesterblom's paper should not be questioned regardless of these references. I have not previously seen such curves as those given by him for the end point and mid point moments at different angles for uniformly distributed load. However, the paper cannot be given priority for the given solution, even though the reader must get the impression from the introduction that the paper deals with the first solution of this problem.

*A. J. Sutton Pippard** (by letter): The author claims that the curved beam has received scant attention. He then deals with the circular arc beam by strain energy methods.

May I point out that this problem has been treated very fully in readily accessible publications.

In 1914 Professor Gibson and Mr. Ritchie published "A Study of the Circular-Arc Bow Girder" (Constable & Co. Ltd., London) in which slope-deflection methods were used to obtain general solutions for both simple and compound arc bow girders.

In 1926 at the request of the Building Research Department of the Department of Scientific and Industrial Research I carried out, in conjunction with F. L. Barrow, a general analysis of this problem.[6]

The method of strain energy analysis was adopted and our treatment was not restricted either to the circular arc girder or to the case of uniformly distributed loading as in Mr. Oesterblom's paper. Instead, the equations were determined in a perfectly general form for a girder of any shape and for any conditions of load. When the girder consists of a circular arc the equations are readily integrable and the results obtained are the same as those given by the Gibson and Ritchie equations. In the general case, however, the equations cannot be integrated directly but require graphical treatment as described with worked examples in the second part of the paper. The main analysis of this paper is also repeated in my book "Strain Energy Methods of Stress Analysis" (Longmans Green & Co., London & New York.) pp. 41-54.

Under these circumstances I think Mr. Oesterblom will concede that his opening sentence is less than just either to myself or to the other authors I have mentioned.

F. W. Dekker† (by letter): Mr. Oesterblom's paper contains several equations and graphs, giving the bending and torsion moments in

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*Professor of Civil Engineering, University of Bristol, England.

[6]"The Stress Analysis of Bow Girders," A. J. Sutton Pippard, and F. L. Barrow, Building Research Tech. Paper No. 1. His Majesty's Stationery Office, 1926.

†Engineer, Amsterdam, Holland.

these beams for values of $\frac{h}{b}$ = 1, 2 and 4 respectively. The equations and graphs for $\frac{h}{b}$ = 1 are the same as those, given by Mr. Wisselink and the writer in *Gewapend Beton*, January 1925,[7] in which it will be noted that for each value of the angle α the bending and torsion moments from our graphs are exactly the same as those of the graphs of Mr. Oesterblom. These formulas and graphs are used with very good results in this country. (Mr. Wisselink died June 22, 1931.)

*W. R. Suda** (by letter): I was glad to see attention given the importance of torsion and I venture the suggestion that although the integrations indicated are no doubt lengthy, they are necessarily a part of the demonstration — could they be published.

The distribution of the torsional shears in the section and methods of reinforcing would make an interesting paper.

Torsion in flat slab construction — end girders framing into columns — the girders are under torsion and column moments may be used to counteract them; torsion in bents, brackets and cantilevers, etc.

Stepping away from torsion another subject that needs presentation is "beam on continuous flexible support."

The old de St. Venant mathematics require more extensive adaptation and so far no material has been so susceptible to the application of the real theory of structural design as is concrete.

AUTHOR'S CLOSURE

I. *Oesterblom* (by letter): It is encouraging to note that several of the participants in the discussion have taken pains to deny the author's priority in the solution of the problem, because this would seem to show that the paper has some merit. Mr. Dekker, Amsterdam, states that the results check with those published by him and his associate, Mr. Wisselink, in 1925, and since then used extensively. Dr Pippard, Bristol, England, says that the opening sentence is not quite just, because others had already treated the subject thoroughly. Dr. Ing. K. Hajnal-Konyi writes from Darmstadt, Germany, that the graphs are something new in the presentation and that the paper must be given credit for practical usefulness.

He denies, however, that priority in solution can be claimed, and quite correctly so. The author's paper was written quite independently and without knowledge of any other similar paper; but he would have

[7]"Buigings-en Wringingsmomenten in Gewapend-Beton Balconen Erkerliggers met Rechthoekige Constante Doorsnede en Cirkevormige – of Gebroken As;" W. J. Wisselink and F. W. Dekker.
*528 N. Brice St., Baltimore, Md.

been bold indeed, if he had thought he would be able to claim priority. With the mass of papers on the subject of structural mechanics the problem could hardly have escaped attention by some authority, but with the complete absence of indexing a few years ago and a merely partial indexing today references would be difficult to find.

The author's cautious reference to "scant attention" should therefore only be taken to mean, as it clearly says, that, in comparison with articles available on the various problems of straight beams and arches, the literature on horizontally curved beams is very limited. This is true without any qualifications, not only for the United States and England, but also for Continental Europe, or any other parts of the world.

Although the author had searched diligently for references, he discovered none until his calculations were finished, when, quite accidentally, he found the two papers by St. Hessler. Later on, while searching for other matter, he likewise found the research paper by Georg Unold. No other papers were found until the discussion disclosed that there were several others.*

This statement, together with the internal evidence of the paper itself, should set at rest the anxiety of Mr. Dekker, of Dr. Hajnal-Konji, and of Dr. Pippard in regard to any priority claims. Without any knowledge if there were or were not similar papers in existence, the author merely attempted to produce a reasonably correct and technically useful paper to provide for the most common problems of design. It is interesting to note that F. Düsterbehn has approached the problem from the same point of view. He has limited his studies to circular curves and to uniform loads, as has the author. These assumptions cover most actual work.

It is to be regretted in response to Mr. Suda that integration of equation (9) can not be given in detail. Many preliminary goniometric transformations would have to be made before the expressions would be in form ready for direct integration; the space required would therefore be prohibitive. We have assurance from Mr. Dekker, however, that the results agree with those obtained in his and Wisselink's paper, used extensively in practice since 1925, also from Dr. Hajnal-Konji that the final equations agree with those of St. Hessler and may check by comparison with the other references as well.

Mr. Suda also comments on faults of brevity and omissions. The author is conscious of both, but these limitations were unavoidable.

*In justice to Mr. Oesterblom it should be recorded that in a letter to the Institute subsequent to the submission of his paper and prior to its publication, he mentioned the St. Hessler and Unold references.—Editor

The paper is merely a beginning of what might be a series of papers, built on the first as a basis—something we hope to attend to later. To work out all possible developments and applications would have extended the paper into a book.

In special answer to Mr. Suda's comments: both the English book by Gibson and Ritchie and the German book by Unold contain extenssive discussions of the distribution of torsional shear stress in a section as well as a mass of experimental verification of theory. These references, previously unknown to the author, should therefore be added to the original references in the paper.

It is to be hoped that the author's formulas and graphs will prove to be useful and also extensively used, but of fully equal value will be the very extensive references called forth, both directly and indirectly, through the discussion. See the following bibliography.

The bibliography contains in sequence names of authors and publications, place and year of publishing, and page number.

F. Grashof, Elastizität und Festigkeit, Berlin, 1875, p. 294.

M. Koenen, Deutsche Bauzeitung, Berlin, 1885, p. 607.

J. Stutz, Zeitschrift der Österreichische Architekten und Ingenieur Verein, Vienna, 1904, p. 682.

K. Federhofer, Zeitschrift für Architektur und Ingenieurwesen, Wiesbaden, 1910, p. 459.

A. H. Gibson, Transactions Royal Society of Edinburgh, 1912, p. 391.

R. Mayer, Zeitschrift für Mathematik und Physik, Leipzig, 1913, p. 246.

B. G. Kannenberg, Der Eisenbau, Leipzig, 1913, p. 329.

H. Müller Breslau, Neuere Methoden der Festigkeitslehre, Leipzig, 1913, p. 258.

K. Federhofer, Zeitschrift für Mathematik und Physik, Leipzig, 1914, p. 48.

A. H. Gibson and E. G. Ritchie, Circular Arc Bow-Girder, London, 1914, New York, 1915—entire book.

H. Marcus, Zentralblatt der Bauverwaltung, Berlin, 1916, p. 501.

W. L. Andrée, Der Eisenbau, Leipzig, 1918, p. 184.

A. Beran, Prager technische Blätter, Prague 1919, Nris 3 & 4.

F. Düsterbehn, Der Eisenbau, Leipzig, 1920, p. 73.

Georg Unold, Forschungsarbeiten, V. D. I., Berlin, 1922, V. 255.

W. J. Wisselink and F. W. Dekker, Gewapend Beton, Amsterdam, 1925, p. 101.

A. J. Sutton Pippard and F. L. Barrow, Building Research Paper, His Majesty's Stationary Office, London, 1926, Bulletin No. 1.

St. Hessler, Beton und Eisen, Berlin, 1927, p. 429.

A. J. Sutton Pippard, Strain Energy Methods of Stress Analysis, London, 1928, p. 41.

St. Hessler, Beton und Eisen, Berlin, 1930, p. 149.

G. Worch, Beton und Eisen, Berlin, 1930, pp. 167, 183, 200.

I. Oesterblom, Journal American Concrete Institute, Detroit, 1932, p. 597.

It has not been possible for the author to see all the references. He wishes to comment on some of them, but such comment should not be taken as implied criticism of those not specially mentioned. Only

a few of the papers are of a sufficiently broad nature, or have carried their solutions far enough, to be immediately useful.

The paper by Wisselink and Dekker is excellent and directly useful, as is that by Düsterbehn, the former treating many polygonal as well as circular frames. Düsterbehn's solution (not his method) is somewhat limited, dealing with semi-circles only, on three, four, and five supports, and with circles on eight supports.

Unold's universal and very comprehensive paper can be recommended for a research student, but not for the general practicing engineer with an American or English education. The same is true about the paper by Marcus.

The two papers by Hessler and the three by Worch should appeal to both the scholar and the engineer. Hessler has solved several important special cases; Worch's solutions are more universal, very simple and directly useful.

Of special interest to American engineers should be the book by Gibson and Ritchie and the Building Research paper developed therefrom by Pippard and Barrow. The first is limited to circular construction, but universal in the matter of extent of arc and type as well as location of loading; the second is universal to the fullest extent. To attain the fullest universality the analytic solutions have been turned into graphic methods, which, for circular projections, by necessity, will mean more labor in design. For unusual curves or loadings one could wish for no better method, however.

Pippard's book on strain energy methods can also be recommended. It is based on Castigliano's classical work, an abbreviation for English students, and contains a chapter on curved beams.

Most of the solutions are based on Castigliano's method, as is that of the author; some have used the elastic weight method by Maxwell and Mohr; others have attempted the use of an elastic curve in three dimensions.

For economy of space the individual titles of the various papers (many long and descriptive) have been omitted. The academic and honorary titles of the authors, for the same reason, have also been omitted.* They all rank high in both respects—authorities of the highest order.

It is fortunate for American engineers that the paper has brought out such a complete bibliography of the subject. Apart from the object of service in submitting it for publication there has been a desire, with so much excellent material available, to invite authors

*Conforming also with the practice of this Journal—Editor

of American text-books to give some little space to the problem in their books. For bay-windows, for balconies in theatres and auditoriums, for silo and tank construction the curved construction is constantly being used, and it is hardly fair to turn out students without some knowledge how these things should be designed.

In the matter of application there is still much work to be done to make the paper fully useful. The question of anchorage and restraint* deserves a special paper, as does the application to silo and tank construction. Mr. Stineman has suggested that the formulas might be modified so as to be applicable for finding windstresses in bridges. By approximating the circular arc to the actual arch of whatever curvature and taking the sections at the quarter points an immediate answer can be obtained, and one close enough for practical design. If more accurate results are needed the Pippard-Barrow graphic method should be used.

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*Mr Lord in a personal letter has very properly suggested that these matters should be dealt with more fully—AUTHOR

The Round-Head Buttress Dam

BY FRED A. NOETZLI*

SYNOPSIS

This paper describes a buttress type of dam in which the water pressure is supported by buttresses whose upstream portion is enlarged laterally into bulb shaped heads joined with the buttress heads of the adjacent units, thus providing a continuous upstream face for the structure. The buttress head of each unit has such a shape that the water pressure is transmitted through it onto the buttress wall by direct compression. The dam functions substantially in the same manner as an ordinary gravity dam, and the design of a buttress unit is in principle similar to that of a vertical slice of a structure of the conventional gravity type.

Typical designs are given for buttress units of 35, 50 and 60-ft. spans along the water face, suitable for dams from 75 to 350 ft. high.

A short description is given of the Don Martin round head buttress dam built in 1928-29 in Mexico.

1. INTRODUCTION

One of the earliest masonry dams of which we have a record, and at the same time the largest masonry dam ever erected, was built about 1700 B. C. for irrigation purposes in Yemen, Arabia. It was about 120 ft. high, 500 ft. thick at the base and 2 miles long. It is estimated to have contained 15,000,000 cu. yd. of stone masonry. It lasted for nearly 2000 years, but was neglected and finally failed after having shown signs of distress for many years before the final collapse.[1]

The early Egyptian and Chaldean engineers appear generally to have followed the rule to make the base of a dam four times its height. The Romans and Moors who built many masonry dams in northern Africa and in Spain made some progress in dam design by reducing the thickness at the base to three times the height.

The French engineers developed in the 19th century the first rational theory of gravity dams. Typical profiles were devised by DeSazilly, Delocre, Bouvier and others on the basis of mathematical principles.

*Consulting Hydraulic Engineer, Los Angeles, Calif.

[1]Some Dams of the Ancients, by Charles Prelini, *Engineering News-Record*, October 6, 1921.

Maurice Lévy was first to introduce the consideration of uplift in the design. In this country Wegmann made during the eighties of the past century his classical studies on gravity dams, and established rules of design which have been used extensively ever since.

From the structural point of view the solid gravity type dam is not a very economical structure, inasmuch as it depends for its stability on its weight alone. In a dam of moderate height the strength of the concrete can not be utilized efficiently. For instance, at the base of a gravity dam 200 ft. high the stresses upon a horizontal plane are about 200 p.s.i. at the downstream side and close to zero at the upstream face. The average is around 100 p.s.i. which is relatively very little for 2000 p.s.i. concrete. Despite these low unit stresses in the concrete the factor of safety against overturning and sliding is little over unity.[2] There are few types of engineering structures of accepted design standard which have as small a factor of safety as an ordinary gravity dam and not many in which the material is used as inefficiently. Some progress has been made during the last two decades in the development of more economical dams of the single arch and buttressed types. Unfortunately, in the endeavor to obtain the maximum degree of economy of material in these "lighter than gravity" type dams some designers in the past undoubtedly have gone too far in the use of thin walls in dams. Such extremes have done much harm and have tended to retard, rather than advance, progress along orderly lines.

2. GENERAL CONSIDERATIONS

The fundamental requirement for a safe and economical dam was expressed admirably by Geo. I. Dillman as follows:[3] "To construct one watertight surface and build the remainder of the structure to support that surface."

In the ordinary type of gravity dam the entire thickness of the section serves for watertightness and at the same time also for supporting the load. This involves an element of strength as well as one of weakness in that the large thickness of the section furnishes for the water a long path of percolation but, unfortunately, the water in the interior of the concrete usually is under pressure and thereby produces undesirable uplift forces in or under the dam.

In buttressed dams of the Ambursen and multiple arch types, sometimes also called "hollow gravity dams," Mr. Dillman's fundamental hydraulic principle finds application in a much clearer and more effective manner than in the uniform section gravity dam. The deck (slabs or arches) of relatively small thickness constitutes the "water-

[2]Safety in Dam Construction, by Allen Hazen, World Engineering Congress, Tokio, 1929.
[3]*Transactions Am. Soc. C. E.*, 1902, p. 95.

tight surface" and the buttresses are designed to support this surface. Herman Schorer has developed a method[4] whereby the quantity of concrete in the buttresses can be made a practical minimum for any desired permissible extent of working stress. Thus, it appears, that reinforced concrete dams of the thin buttress type require practically the minimum quantity of material of which concrete dams can be built for given conditions. In general the quantity of concrete in these dams is only 25 to 35 per cent of that of an ordinary gravity dam, but, of course, the cost per cu. yd. of the reinforced concrete structures is at least twice as great as the cost of the mass concrete of the uniform section type.

In the following chapters there is given a description of an intermediate type of gravity dam which, to some extent, combines the advantages of mass concrete construction of the uniform section gravity dam with the economy of buttressed types.

3. TYPICAL FEATURES OF ROUND-HEAD BUTTRESS DAM

The type of dam to which special reference is taken in this paper consists of a series of buttress units, each having towards the water side an enlarged bulb-shaped, buttress head joined laterally with the heads of the adjacent units in a manner as to form a continuous upstream face of the dam. The buttress head of each unit has such a shape that the water pressure is transmitted to the buttress wall proper by direct compression. In the typical round-head buttress unit this is accomplished by making the upstream face curved convex toward the water side, as shown in Fig. 1 in the simplest theoretical shape. A similar result may be obtained by constructing the upstream face of polygonal shape, for instance, such as shown in Fig. 2. For convenience this may be called the "diamond" shaped buttress head.[5] The area of contact between adjoining heads may be made as large as desired, within reason, in the manner indicated in these two figures.

One of the important features of the round-head buttress type of dam is the shape of the upstream face of the buttress heads. The water pressure is transmitted by direct compression upon the buttress walls. There are no lateral bending moments or arch stresses in the buttress heads and no steel reinforcement is required.

Inasmuch as the water pressure acts perpendicularly to each surface element the shape of the upstream face should be determined for sections perpendicular to the upstream slope of the dam. The shape of the buttress head in horizontal sections then can be readily determined.

[4] *Proceedings, Am. Soc. C. E.*, Nov. 1930, Papers and Discussions, p. 1947.
[5] See description of Don Martin Dam by C. H. Howell, *Am. Soc. C. E. Transactions*, Vol. 96, 1932, p. 857.

The relatively large mass of the rounded buttress heads is beneficial in several ways. First, it is in the position to resist most efficiently the overturning moment of the water by weighing down the upstream portion of the dam. Second, it places the largest cross section of the dam toward the water face, thus offering a maximum length of path to water percolating through construction joints or pores in the concrete. Third, it reduces the danger of damage to the concrete due to freezing. Fourth, it improves the size and distribution of the inclined principal stresses in the buttresses, as demonstrated in the excellent paper by C. V. Davis, entitled "Recent Advances in Buttress Type Dams."[6]

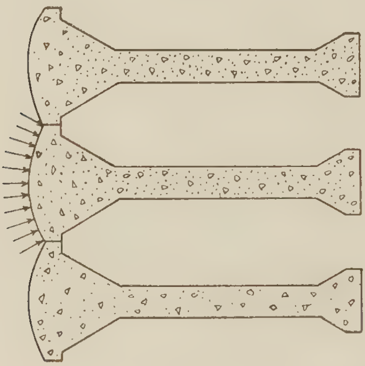

Fig. 1

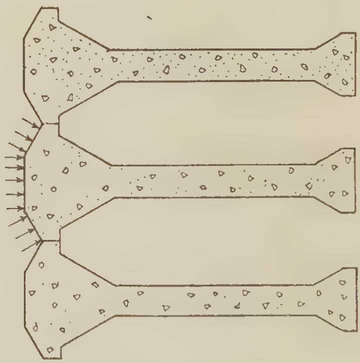

Fig. 2

By a large Tee on the downstream end of the buttress walls material can be provided where it will work under the maximum unit stresses and therefore with the greatest efficiency when the reservoir is full. Thus, the upstream head is useful by its weight, and a large downstream Tee is efficient in stress for transmitting the water load onto the foundation. The buttress wall will then function mainly as a web designed to connect the "A"-frame formed by the buttress head and the down-stream Tee. Of course, the buttress web must be dimensioned so as to transmit safely the shear and other stresses between the two principal members of a buttress unit, but from the structural point of view the function of the web is of secondary, rather than of primary importance. The web is, therefore, the place where the designer may effect the maximum economy of material by making it as thin as the analysis of stresses will permit. It should be noted that the web of the round-head buttress dam corresponds to the interior portion of an

[6]*Civil Engineering*, February 1931, p. 387.

ordinary gravity dam. In this portion of a gravity dam the compression stresses are always low, and a saving of concrete as effected in the round-head buttress dam and as indicated by Fig. 3, is therefore feasible without in any way decreasing the safety of the structure.

The individual buttress units are structurally entirely independent of each other. The failure of one buttress is therefore much less likely to affect the stability of the adjoining parts of the dam than might be the case, for instance, in a multiple arch dam. The design of a buttress unit is similar to the design of a vertical slice of a gravity dam, the main difference resulting from the somewhat irregular shape of the

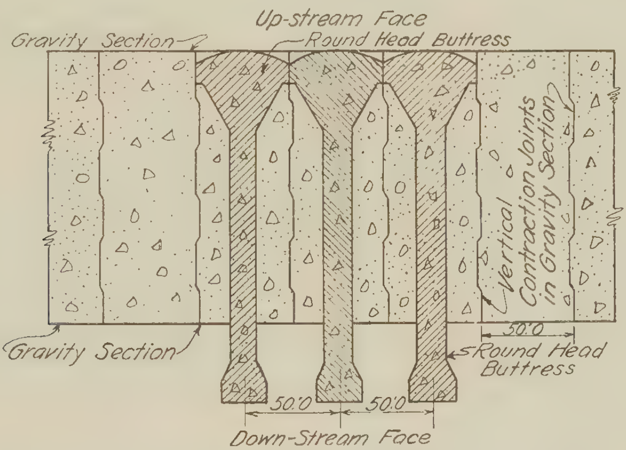

FIG. 3—TYPICAL HORIZONTAL SECTION THROUGH GRAVITY DAM AND THE CORRESPONDING CROSS SECTION OF A ROUND HEAD BUTTRESS TYPE

horizontal sections, and further, from the fact that on account of the enlarged buttress head a greater width of water is carried by a buttress than that corresponding to the thickness of the buttress wall alone.

The upstream face of the dam is on a slope to obtain a sliding factor of permissible size, generally berween 0.65 and 0.75. Usually, a trench of suitable depth is excavated into the foundation rock, and the concrete for the buttress foundation is puddled into this trench to fill it completely, thus providing an area of contact and bond between concrete and bed rock not only at the bottom of the trench but also along the rough side walls. Also, the enlarged buttress head offers additional wedge-like resistance against sliding, so that the sliding of a buttress out of such a trench foundation seems hardly conceivable. Above the foundation a sliding in the joints between daily pours can be prevented

by steps and keys or other suitable construction methods. It is now generally recognized that in buttressed dams the concrete may be assumed to resist safely a moderate amount of shear also in the horizontal construction joints.

4. TYPICAL DESIGN TYPES

The round head buttress type of dam is well adapted to the use of typical design types. No attempt will be made to generalize on the determination of the most economical spacing of buttresses. Besides theoretical quantity considerations this question depends largely on the relative cost of cement, aggregate, forms and construction plant. Such studies are preferably made individually for each dam site. In some cases the deepest portion of the dam site may be closed by a few relatively high units, while on one or both wings the dam will be relatively low. In such a case the greatest economy may be obtained by a combination of buttress units of different spans.

Wherever the aggregate is readily available and relatively cheap it generally will be found that a larger spacing of buttress units is more economical than a smaller spacing, although the latter will naturally require less concrete. Whenever there would appear to be little, if any difference in the total cost of the dam for a large or a smaller spacing of buttress units the writer would favor the larger spacing on account of the greater thickness of the sections. In case of overflow dams the economic spacing will be largely influenced by the best suited span of the downstream deck slab.

Three typical designs and layouts of round head buttress types are presented herewith having spans of 35, 50 and 60 ft. It is believed that these designs will be suitable for practically the entire range of height to which dams of this type may be considered for the present. A general study made by the writer indicated that the three types will be found well suited for the following conditions:

(1) 35 ft. spacing for dams 75 ft. to 150 ft. high
(2) 50 ft. spacing for dams 150 ft. to 250 ft. high
(3) 60 ft. spacing for dams over 250 ft. high.

A. Typical Design with 35 Ft. Spacing of Buttresses

Fig. 4(a) shows a typical layout of a buttress unit of 35 ft. span and 150 ft. height. The upstream face of the buttress heads is circular in horizontal planes. Normal sections are therefore slightly elliptical. One might also make the normal sections circular so that the horizontal sections would be elliptical, but it is simpler to lay out the forms for circular horizontal sections. The upstream radius in horizontal planes is 30.5 ft. The side faces of the buttress heads converge on a

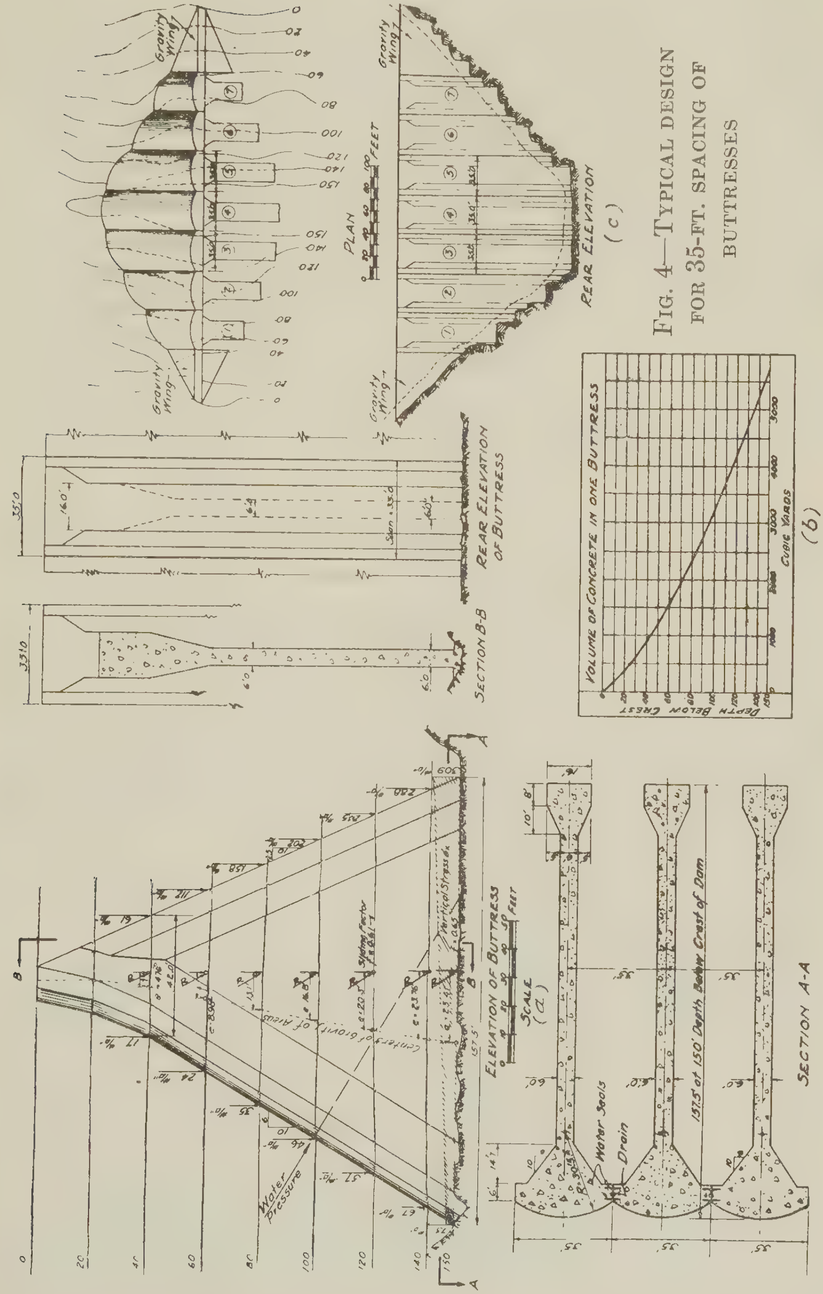

Fig. 4—Typical Design for 35-ft. Spacing of Buttresses

batter of 6:10 measured in a horizontal plane. This batter is somewhat flatter than that corresponding to the 30.5 ft. radius of the upstream face, but is required in order to produce radial transmission of the water pressure in sections normal to the upstream face. Several factors besides stress conditions were considered in the design which resulted in the shape of the buttress unit shown in Fig. 4(a). A Tee of large dimensions is provided along the downstream side for the purpose of stiffening the buttress wall; also to have a large concrete section where the vertical stresses are greatest and where consequently the concrete works with the highest efficiency. The thickness of the buttress wall was arbitrarily chosen at 6.0 ft. from top to bottom, as this thickness was considered the minimum for mass concrete construction of this type. The thickness of the buttress wall, and the shape of the buttress head and of the buttress Tee are the same from the base of the buttress to a short distance below the crest. The form work is therefore relatively simple and inexpensive.

The upstream face of the dam is on a slope. This produces two most desirable results,—first, the water overlying the up-stream face with full reservoir provides automatically and without cost the weight necessary for a safe sliding factor; and second, the water pressure on the upstream face is acting in a direction which is inclined downwardly towards the base of the dam and intersects the same well within the interior of the section. The dam has therefore a large factor of safety against overturning. Any increase of the water pressure, or silt pressure, would merely tend to press the dam more firmly upon the rock foundation. This feature gives this type of dam a decided advantage over the ordinary gravity section with its more or less vertical up-stream face.

The principal dimensions, vertical stresses and sliding factors for a buttress unit of 150 ft. height are shown in Fig. 4(a). At the base the vertical stresses are 73 and 309 p.s.i. at the up-stream and down-stream faces, respectively. The maximum sliding factor at the base is $f = 0.65$.

A buttress unit of 35 ft. span along the up-stream face, and of a height of 150 ft., as shown in Fig. 4(a), contains 5800 cu. yds. of mass concrete. A portion of a gravity dam of the same length and height, having a vertical up-stream face and a down-stream batter of 0.73 would contain about 10,700 cu. yds. The difference is 4900 cu. yds., or about 46 per cent. For lower units the difference is somewhat less.

The curve of Fig. 4(b) gives the quantities of concrete of a buttress unit for different heights below the crest. This curve is very convenient for estimating purposes. Thus, in order to determine approximately

the quantity of concrete in an entire dam, draw the center lines of the buttress units (35 ft. apart) on the profile of the dam site. The height of each buttress above the assumed line of foundation rock is scaled from this profile, and the quantity of concrete in each buttress unit is read directly from the quantity curves of Fig. 4(b). The total sum is evidently equal to the total quantity of concrete of the entire dam.

At most dam sites the height of the dam at the wings is relatively small. It may then be advisable to build these low wings of ordinary gravity section as indicated in Fig. 4(c). As an example there is given in Table 1 an estimate of quantity of a dam of about 155 ft. height shown in Fig. 4(c) by a typical plan and elevation. The dam consists of 7 buttress units and two low gravity wings.

TABLE 1

Buttress No.	Height Ft.	Volume Cu. Yds.
(1)	(2)	(3)
1	90	2700
2	134	4900
3	154	6100
4	155	6200
5	153	6000
6	115	3900
7	85	2500
2 gravity wings		2500
Total concrete in dam		34800 cu. yd.

For each buttress unit (col. 1) the height of the buttress (col. 2) is scaled from the profile of the dam site or estimated from the contour lines, making due allowance for the estimated depth of excavation. The quantity of concrete in each unit (col. 3) is then readily obtained from the quantity curve of Fig. 4(b). Finally, the volume of the two gravity wings is computed by any convenient method. The total sum of the values in col. 3 is the estimated quantity of the dam.

In the present example the round head buttress dam was estimated to contain 34,800 cu. yds. of concrete. A gravity dam at the same site was estimated at about 58,000 cu. yds. The difference is 23,200 cu. yds. or approximately 40 per cent for the entire dam.

B. Typical Design for 50 ft. Spacing of Buttresses

A typical layout of a buttress unit of 50 ft. span is shown in Fig. 5(a). As stated previously, it is believed that this span is best suited for dams from 150 to 250 ft. high. However, under certain conditions it may be well adapted also for dams only 125 ft. high or as high as 300 ft. The dimensions, vertical stresses and sliding factors of a typical unit are all shown in Fig. 5(a).

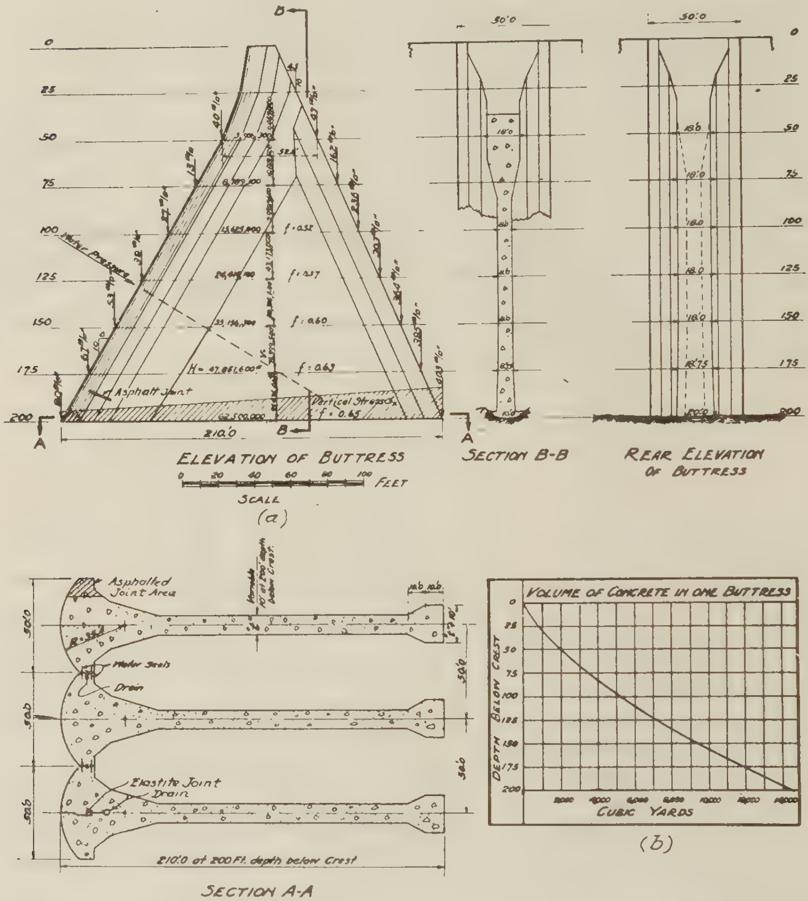

FIG. 5—TYPICAL DESIGN FOR 50-FT. SPACING AT BUTTRESSES

The quantity of concrete in a buttress unit of 50 ft. span is given by the diagram of Fig. 5(b) from which the quantities of a dam may be estimated in a similar way as explained for the 35 ft. span.

In a round head buttress dam with units of 50 ft. spans the joints between the units are spaced at the same distance from each other as are the vertical contraction joints in many gravity dams (50 ft.). If desired, an additional contraction joint may be provided in the plane of the center line of the buttress heads, thus reducing the distance between joints to 25 ft. Such a joint in the buttress head is indicated in Fig. 5(a). It extends only part way through the heavier section

of the buttress head to a point from which on the shrinkage of the concrete in a lateral direction is evidently negligible. Such a joint may be formed by placing in each lift of concrete, in advance of pouring, a sheet of Elastite or other material in the plane of the joint and holding it properly in position while pouring concrete simultaneously on both sides of the sheet. There must be a water stop across the joint along the up-stream face, and a drain in the interior of the buttress head to prevent water under pressure from entering the joints.

There are other methods available for decreasing undesirable effects of shrinkage in the buttress heads. One such method is illustrated by Fig. 5(a). It consists of providing asphalted joints extending in normal planes part way from both sides of a buttress head. Thus, the length of concrete fixed to a relatively immovable base may be decreased as much as desired, so that there is no likelihood of shrinkage cracks developing in the solid middle portion of the buttress head. To be most effective the lowest of these lateral joints should be placed a very short distance above the bed rock. If desired, other such joints may be placed at suitable intervals higher up in the buttress heads. They must be sealed by water stops or otherwise, to prevent leakage. The joints should preferably be in planes approximately normal to the up-stream slope. In this position they will not weaken the structure in any way. In fact, inasmuch as practically no stress will be transmitted across the area of these asphalted joints it will be proper to reduce in the design the horizontal sections through the dam by these areas, thus shifting the centers of gravity down stream and decreasing the eccentricity of the load. This will improve materially the distribution of stresses in the buttress and is therefore structurally a decided improvement.

The relatively thick buttress walls of a round head buttress dam, in conjunction with the stiffening effect of the enlarged buttress heads and down-stream Tee will in general require little, if any, lateral bracing. For instance, in a dam 200 ft. high with a spacing of the buttresses of 50 ft., the column ratio, that is, the proportion between lateral thickness of the buttress head to the height of the dam is $50:200 = \frac{1}{4}$. For the lower units of the same dam the vertical column ratio is even more favorable. For the down-stream Tee the column ratio may be easily kept as low as $\frac{1}{8}$.

C. Typical Design for 60 ft. Spacing of Buttresses

For dams of considerable height, say, over 250 ft., it will in general be desirable to use buttress units of longer span and larger cross section of the buttress walls.

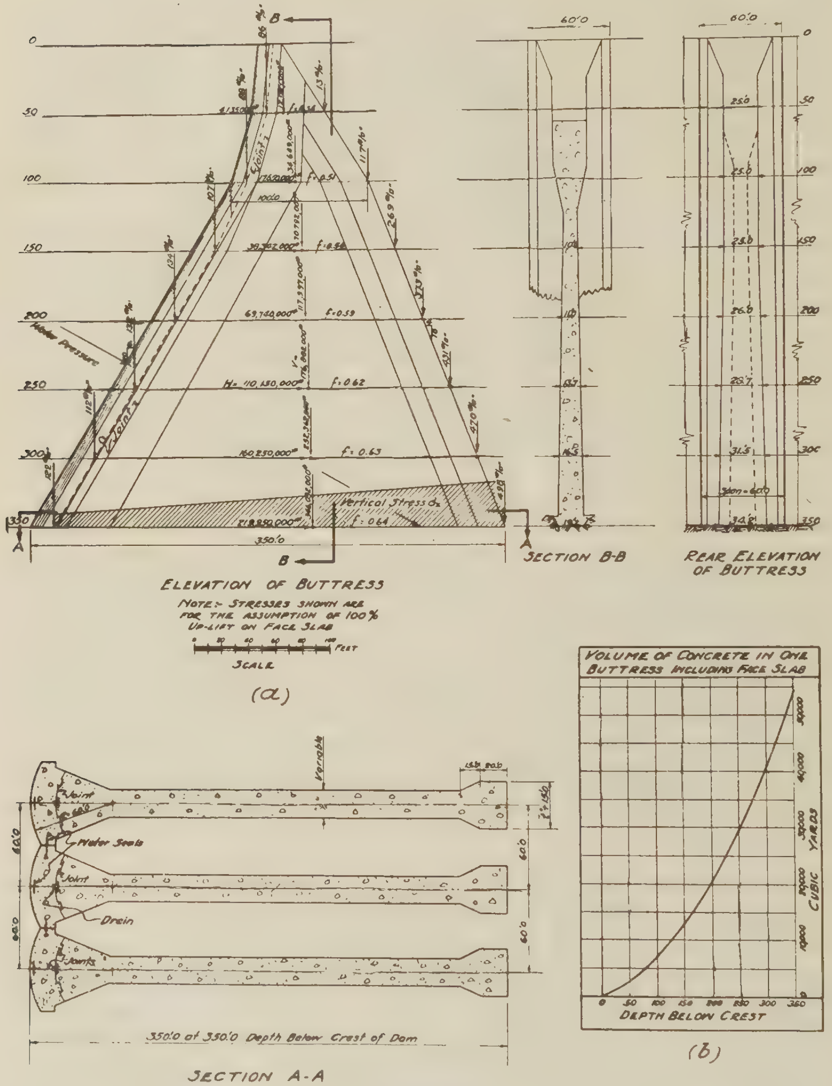

FIG. 6—TYPICAL DESIGN FOR 60-FT. SPACING OF BUTTRESSES

Fig. 6(a) shows a typical layout of a buttress unit of 60 ft. span. The design is for a unit of 350 ft. height, but could be extended to greater heights, if desired. The buttress wall has a minimum thickness of 10 ft. At the base of a unit 350 ft. high the thickness is 19.2 ft.

The buttress head is jointed lengthwise and crosswise in order to provide for shrinkage of the relatively large mass of concrete in this portion of the dam. At the junction of the two joints there is a well of large dimensions for drainage and inspection. This well also serves for cooling the concrete in the interior of the buttress head during the process of hardening of the cement. Toward the crest of the dam the transverse joint in the buttress head is placed relatively close to the up-stream face in order to provide in the buttress section proper a sufficiently large cross section for supporting the water pressure. In this portion of the dam the jointed parts serve to some extent as waterproofing face slabs.

An additional important feature of the jointed buttress head is its greater degree of flexibility in case of earthquake movements or uneven settling of the foundation.

The stresses in a buttress unit as shown in Fig. 6(a) were determined under the following three assumptions:

1. The weight and area of cross section of the upstream portions of the jointed buttress head were neglected except for the weight component normal to the face of the buttress which was assumed to produce an overturning moment in the same direction as the water pressure.

2. The up-stream portions of the jointed buttress head were assumed to be interlocked across the transverse joint by key and groove connections in such a way that the full weight of these portions would act upon the buttress. There was further assumed to exist 100 per cent uplift pressure over the entire horizontal area of these up-stream portions.

3. The whole weight of the upstream portions was assumed to be supported by the buttress by means of key and groove connections in the joint but otherwise the cross sectional area of these portions was entirely neglected in the analysis of stresses in the buttress.

Table 2 gives the vertical stresses in a buttress, at upstream and downstream faces, for the three assumptions as outlined above. The figures indicate that none of the three assumptions leads to dangerous stresses. As a matter of fact, it is believed that neither of the assumed extreme conditions will ever obtain in a dam, but that the jointed up-stream portion of the buttress head will participate at least to some extent in supporting the load; so that the dimensions shown in Fig. 6(a) are believed to be ample. If desired, the vertical stresses at the down-stream face may be decreased by lengthen-

TABLE 2

Elevation Below Crest Ft.	1st Assumption Vertical Stress		2nd Assumption Vertical Stress		3rd Assumption Vertical Stress	
	Up-stream	Down-stream	Up-stream	Down-stream	Up-stream	Down-stream
	p.s.i.	p.s.i.	p.s.i.	p.s.i.	p.s.i.	p.s.i.
50	0	80	76	13	130	—32
100	17	159	88	117	138	86
150	48	292	107	269	187	232
200	68	397	134	373	231	331
250	64	454	132	431	228	386
300	53	493	111	470	192	433
350	64	522	122	498	200	462

ing the buttresses. For instance, a lengthening by 5 per cent will decrease the down-stream stresses nearly 10 per cent.

The several stress analyses were made with the object in view of investigating for widely varying conditions the stability of a structure with a relatively flexible buttress head. This type of head may be feasible for spans of 70 ft. to 80 ft., thus affording the use of very thick buttress walls and massive construction of other parts of the dam which may be desirable under certain conditions especially in the case of very high dams.

As an alternative to the jointed buttress head as illustrated in Fig. 6(a) a construction involving joints such as shown in Fig. 5(a) may be used to decrease the effect of shrinkage in the buttress heads.

The quantity of concrete in a dam with 60 ft. spans may be estimated by means of the quantity curve shown in Fig. 6(b) in a similar manner as described in detail on page 166. A 60 ft. unit, 350 ft. high, requires 54,300 cu. yd. of concrete as compared to 99,000 cu. yd. in a uniform section gravity dam with a 0.73 batter.

5. UP-LIFT PRESSURE

Concrete dams are subject to uplift pressure on account of water that is forced from the reservoir either into the joint between the foundation rock and the dam, or into joints between successive pours of concrete. Pressure in the pores of the concrete is in general of less consequence than uplift in joints. In some cases there may also be uplift in the joints in horizontally jointed or fractured bed rock. On account of this uplift pressure it is customary to require that a uniform section gravity dam be designed for a certain amount of uplift ranging according to conditions from 33 to 100 per cent of the full head at the upstream face and diminishing at a uniform rate to zero or to the elevation of the tailwater at the down-stream face. It is a generally accepted fact that buttressed dams are much less subject to uplift pressure than the uniform section gravity type. Usually, the buttresses are

founded into trenches excavated from the rock. Water which might seep from the reservoir under the base of a buttress could readily escape on either side of the buttress. The same holds true of water that may be forced into horizontal construction joints. There may be uplift under or in the heads of a round head buttressed dam, but this is not likely to lead to critical overturning conditions on account of the relatively great length of the buttress walls.

It was shown previously that in a 60 ft. span unit as shown in Fig. 6(a), uplift pressure over part of the entire area of the jointed portions upstream of the transverse joint in the buttress head would not affect very materially the stresses in the buttress walls. The transverse joint and drainage provisions will prevent water from being forced under pressure into the buttress walls in a manner as to produce uplift therein. A round head buttress dam may therefore be considered to be affected by uplift much less than the ordinary type of gravity dam.

6. PRINCIPAL STRESSES

In a dam under water load there occur at all points stresses in every direction which are related to each other according to the so-called "ellipse of stress." The major and minor axes of this ellipse represent the so-called "principal stresses" at the point under consideration. The major axis represents the maximum, often also called, first principal stress, and the minor axis corresponds to the minimum, or second principal stress. The two principal stresses are perpendicular to each other. Ordinarily, the direction of the principal stresses in a dam is inclined with the horizontal plane.

In the older methods of design of dams only the stresses acting vertically upon a horizontal plane were considered. These vertical stresses, also called "trapezoidal" stresses, on account of the shape of their assumed distribution over a horizontal plane are in general not the maximum stresses. In dams in which the stresses are of considerable magnitude it is important to know not only the extent of the vertical trapezoidal stresses, but also the extent and direction of the principal normal and shear stresses. This is particularly necessary for dams of the buttressed type.

The theory of the principal stresses is given in many textbooks.[7] An application to dams and approximate methods of determining the principal normal and shear stresses in buttresses may be found in the writer's chapter on Multiple Arch Dams in Wegmann's "The Design and Construction of Dams."[8]

[7]For instance in "Strength of Materials," by G. F. Swain.
[8]Eighth Edition, 1927, p. 454.

7. INCLINED JOINTS IN BUTTRESSES

In a concrete dam founded on rock the restraint of the foundation prevents the contraction of the concrete when it shrinks due to loss of chemical heat or to drying out. If the dimension of a dam in an upstream-downstream direction is more than a certain amount the shrinkage may produce stresses in the concrete which exceed its tensile strength.

Vertical and more or less inclined shrinkage cracks have occurred in the buttresses of many buttressed dams. A notable example is the Lake Hodges multiple arch dam built in 1917 on the San Dieguito River near San Diego, California, in which inclined cracks opened as much as ⅛ inch. Similar shrinkage cracks have also been observed in the interior of a number of gravity dams[9] and prèsumably also exist in others, although they can not be seen on account of lack of means of access to the interior of these dams.[10]

Horizontal steel reinforcement may help to distribute the cracks in buttresses, but ordinarily can not prevent them. Even in dams where no cracks have developed there will exist considerable tension stresses due to shrinkage of the concrete.

In buttressed dams with flat upstream slopes and relatively thin buttresses the minimum principal stresses parallel to the upstream face may be tension, thus increasing the tendency of cracks developing in such buttresses. The extent and distribution of the principal stresses in such a buttress are much improved by making the upstream slope steeper. In a buttress of approximately equilateral triangular shape the stress distribution is well near ideal. Fortunately the shape of the buttresses of round head buttress dams can be made to approach this ideal shape closely, as will be observed in Fig. 4(a), 5(a) and 6(a).

The distribution of the principal stresses indicate the feasibility of locating in the buttresses contraction joints parallel to the directions of the principal stresses σ_{I} or σ_{II}. In these planes the shear is zero. Therefore, if a joint is located in a buttress parallel to the direction of the first principal stress there is no tendency of portions of the buttress on opposite sides of such a joint moving relatively to each other. In other words, the stresses in the buttress are not changed materially unless the second principal stress acting in the direction perpendicular to the joint should be tension. Properly arranged inclined contraction joints will enable the concrete to contract or expand without the danger of irregular cracks occurring in the buttress.

[9]*Annales des Ponts et Chaussees*, March-April 1930, p. 144.

[10]The plans of the Hoover Dam provide contraction joints in the dam approximately 50 ft. apart, both in transverse and in longitudinal direction. After shrinkage has taken place the joints will be grouted.

When the reservoir is only partly full there will be small shearing stresses in the planes of the joints. It is advisable to provide shear keys or offsets in the joints to care for these stresses.

It might be feasible to locate contraction joints parallel to the second principal stresses, that is, normal to the joints assumed previously. Such an arrangement, however, would weaken a buttress against buckling, unless every unit between joints is braced adequately. Furthermore, the first principal stresses would be transmitted across the joints so that after a dam is built all the joints would be tightly closed. This would obviously defeat the purpose of these joints, namely, to facilitate expansion and contraction of the concrete. It is believed that inclined joints parallel to the direction of the maximum principal stresses which divide a buttress into two or more inclined columns supporting substantially independently of each other their proportion of the water load, are best adapted to buttresses of dams. A dam in which there are joints parallel both to the direction of the first and second principal stresses was designed by James Girand and is under construction in Chile.[11]

8. SPECIAL TYPES OF BUTTRESS HEAD DAM

For some distance downward from the crest of a round head buttress dam the concrete in the buttress heads is not put to very efficient use and the stresses in these parts of such a dam are relatively small. Much economy of material and some saving in cost may be obtained by decreasing the size of the buttress heads in the upper portions and bridging the gaps between neighboring buttresses with short slabs or arches.[12] Suggestions for such composite types of construction are given in Fig. 7 and 8. Of course, it would also be feasible to omit the buttress heads entirely and construct an arch for the whole span. This latter procedure, however, would necessitate long-span arch centering near the crest of a dam where the cost of the forms per cu. yd. of concrete is particularly high and thus might offset any economic advantage.

In a cellular gravity dam, as illustrated in Fig. 9 in a typical horizontal section, the water pressure has a tendency to produce tension in a lateral direction in the cantilevering parts of the up-stream portion of the dam. Such tension stresses can be eliminated by lateral pressure of the water in the area of the joint up-stream of the water seal. In other words, it is only necessary to set the water seal across the slot back from the up-stream face for such a distance that the lateral pressure of the water in the slot produces a negative bending moment of sufficient size to eliminate the tension stresses in the lateral cantilevers. There

[11] *Proceedings, Am. Soc. C. E.*, Jan. 1931, Papers and Discussions, p. 173.
[12] Recent Advances in Buttress Type Dams, by C. V. Davis, *Civil Engineering*, February 1931, p. 387.

probably always will be at least a film of water in the joint up-stream of the water stop. To insure positive action, however, an open slot as shown in Fig. 9 may be provided in the plane of the joint, thus admitting water pressure freely in the plane between adjoining buttress heads.

9. WATERTIGHT JOINTS BETWEEN BUTTRESS HEADS

It is necessary, of course, to prevent undue leakage in the joints between adjoining buttress heads. No particular difficulty has been experienced to make joints in other types of dams, for instance, in gravity, arch and Ambursen dams reasonably watertight.

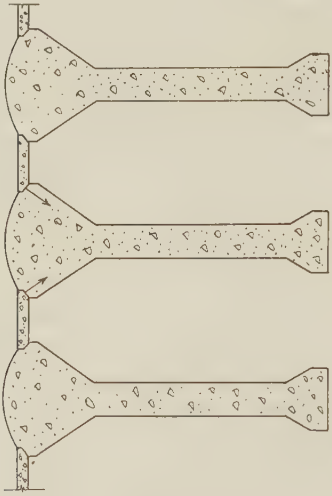

FIG. 7

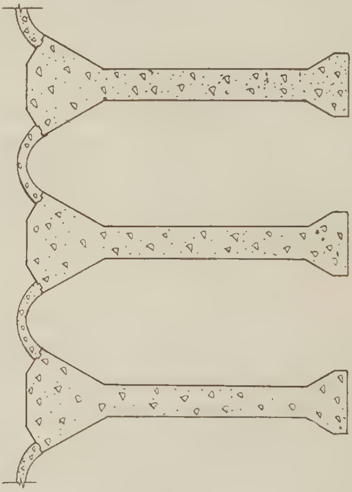

FIG. 8

Usually a suitably bent sheet of copper is placed across the joint. Any water that may pass through or around the seal is then best allowed to flow away through drains or along the joint. A great thickness of the concrete section is therefore not likely to have much influence upon the watertightness of a joint properly sealed near the up-stream face.

In a round head buttress dam a considerable area of contact between adjoining buttress heads can be obtained by the addition to the theoretical buttress head of small triangular bodies of concrete, as indicated in Fig. 1 and 2.

For additional safety against leakage two copper seals may be placed across each joint instead of only one. A drain hole between the two seals would carry away such water as might pass through or around the seal farthest up-stream. Such an arrangement is shown in Fig. 4(a).

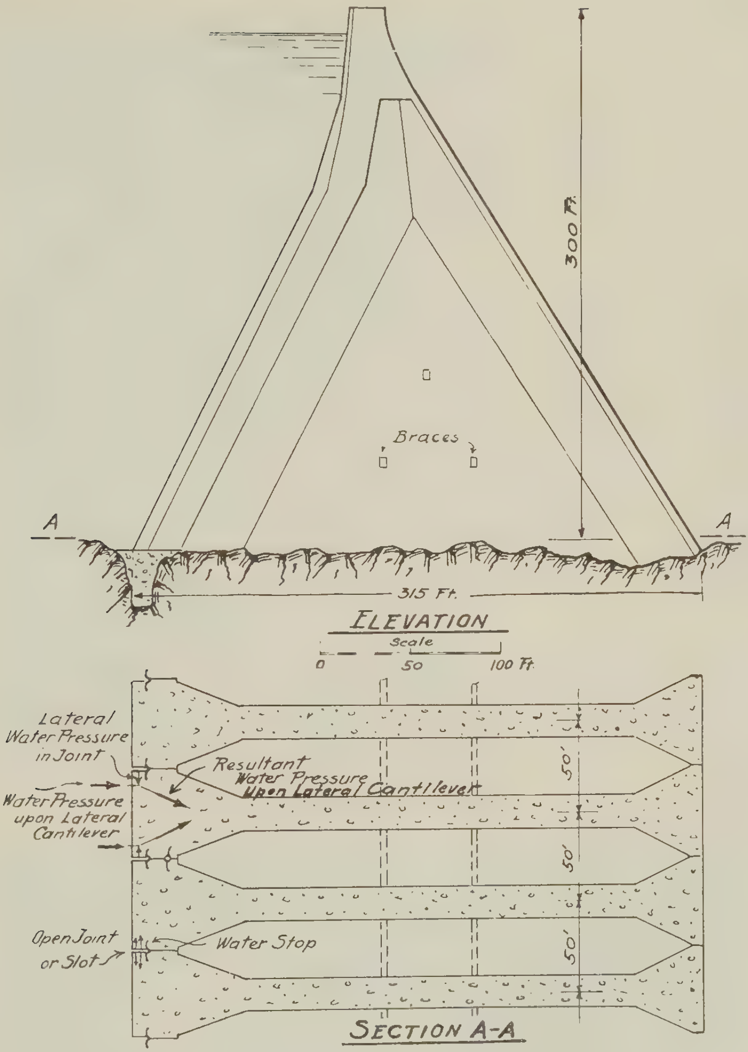

FIG. 9—CELLULAR GRAVITY TYPE DAM

In a round head buttress dam recently constructed a single copper seal combined with a drain has given entirely satisfactory results.[13]

The most effective protection of a round head buttress dam against leakage in the joints as well as through the concrete is a continuous

[13]In a discussion of the Don Martin round-head buttress dam, C. H. Howell, states "To date there has been no measurable leakage between the units. The copper seals between them have proved to be practically watertight." *Proceedings, Am. Soc. C. E.* March 1931, Papers and Discussions, p. 505.

face slab, say, from 1 to 2 ft. thick, extending over the entire up-stream face of the dam and, in connection with a cut-off wall, well down into the foundation. In such a case the upstream face of the buttress heads should be waterproofed with an asphaltic paint or membrane, and drains should be provided to carry away any water that might seep through the face slab. Instead of a concrete slab, it might be preferable in some cases to use gunite.

10. OVERFLOW DAMS

The round head buttress type is well suited for overflow conditions. In fact the first dam of this type ever built, the Don Martin dam in Mexico, is a structure designed for 21 ft. depth of overflow. The shape of the buttresses of the dam, especially the slope of the downstream face can be made to conform to that corresponding to the nappe so that little extra concrete is required except for protection against erosion at the base of the dam. The downstream Tee of the buttresses decreases the free span of the downstream slab so that an economical construction is feasible even for a relatively large spacing of the buttresses.

11. DON MARTIN ROUND HEAD BUTTRESS DAM

The first dam of the round head buttress type was built in 1928-29 for the Mexican Government to store water for the Nueva Espana irrigation project on the Rio Salado, about 60 miles west of Laredo, Texas.[14]

The dam consists of a buttressed section 767 ft. long forming the spillway of an earth dam about 3500 ft. long, which, together with a separate low earth dike more than six miles long creates a reservoir of 1,136,000 acre ft. capacity.

The round head buttress portion of the dam is composed of 26 buttress units. It has a maximum height of 130 ft.

The buttresses are spaced 9.0 meters (29 ft. 6 in.) on centers. The thickness of the buttress walls is 6 ft. 6 in. from top to bottom. There is no steel reinforcement either in the buttress heads nor in the buttress walls. The buttress heads are joined loosely along an area of contact of 7 ft. 9 in. width.

A reinforced concrete slab 2 ft. 6 in. thick is spanned between the down-stream ends of the buttresses to provide for overflow. The storage level in the reservoir is raised 14.5 ft. by means of a series of 26 gates on top of the dam. Of these gates 22 are of the automatic radial type and four are designed for mechanical operation. The maximum dis-

[14]See *Eng. News.-Record*, March 21, 1929; *Western Construction News*, Jan. 25, 1930; *Transactions, Am. Soc. C. E.*, Vol. 96, 1932, p. 841.

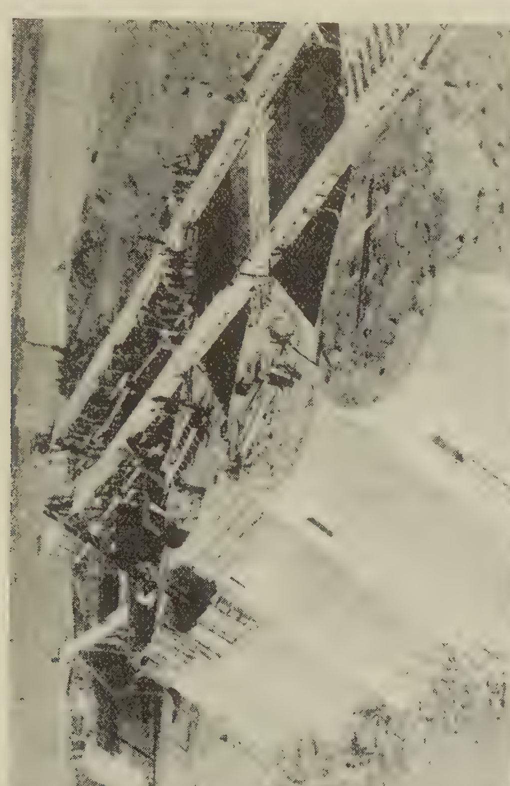

FIG. 10—DON MARTIN DAM, MEXICO. BUTTRESS HEADS WITH CIRCULAR UP-STREAM FACE

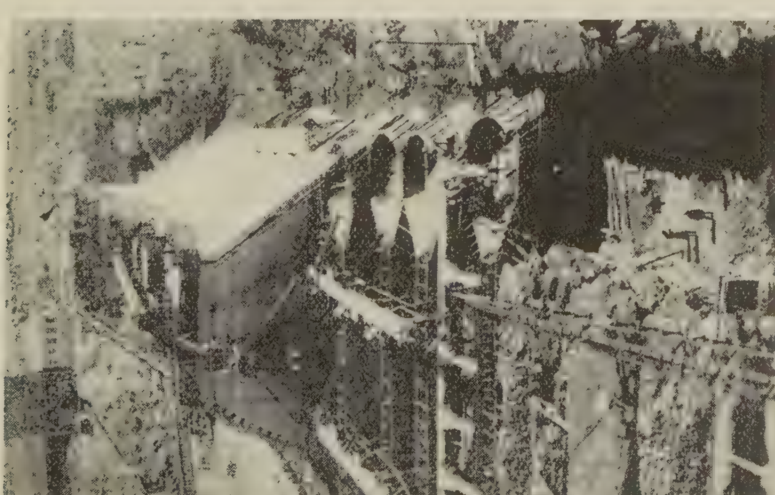

FIG. 11—DON MARTIN DAM, SHOWING DIFFERENT STAGES OF CONSTRUCTION OF BUTTRESS UNITS

charge over the dam with all the gates open and with the water 21 ft. deep on the crest, is 210,000 cu. ft. per sec.

It was estimated that the round head buttress type made possible a saving of about $400,000 as compared to the cost of a gravity dam.

The buttressed section of the Don Martin Dam was designed by C. H. Howell and Julian Hinds, under the direction of F. E. Weymouth.

12. CONCLUSIONS

Particular emphasis was laid in this paper on the description of features of dams of the buttressed type which would permit of a considerable economy of material while still being of mass concrete construction. Other modified types of gravity dams showing various degrees of economy have been described previously.[15] It is quite evident that the ordinary type of uniform section gravity dam involves a relative maximum and the reinforced concrete types a relative minimum of material. It may not always be desirable to select the cheapest type of dam for a given site Nor does there appear to be any good reason for choosing the most expensive type. If a round head buttress dam were designed without attempting to effect any economy over an ordinary gravity dam the round head buttress type could be built of such liberal dimensions that its factor of safety at equal cost would be at least twice that of the ordinary gravity type.

The round head buttress type of dam as described in this paper is believed to possess a number of features which tend to promote both economy and safety. Most important among these features are the following:

1. The dam is of the mass concrete type and can be built of the same class of concrete as is used in ordinary gravity dams.
2. There is no arch, beam or slab action involved in the buttress heads, and consequently, no reinforcement is required.
3. The individual buttress units are structurally entirely independent of each other. The dam is therefore less likely to suffer in case of uneven settling of the foundation, earthquake movements or other causes.
4. Simplicity of design, similar to that of an ordinary gravity dam.
5. Uplift pressure is not likely to endanger seriously the stability of a round head buttress dam even under very unfavorable conditions.
6. The sliding of a buttress unit out of a properly prepared rock trench would appear to be very improbable. In the concrete above the base the danger of sliding can be practically eliminated by step-offs and keys in the joints between consecutive lifts of concrete.

[15]Modified Types of Gravity Dams in Relation to Uplift, *Eng. News-Record*, Dec. 4, 1930, p. 884.

7. By virtue of the sloping up-stream face of the dam the pressure of the water is directed downwardly towards the base of the dam. This results in a large factor of safety against overturning. Furthermore, silt in the reservoir will tend to "weigh down" the heel of the dam and thereby decrease the overturning moment.

8. The combination of a buttress head with a large down-stream Tee, connected by a relatively thin web permits a very efficient use of the concrete. The large mass of the buttress head tends to weigh down the up-stream portion of the dam, and the Tee contains the concrete which serves most efficiently to transmit the water load to the foundation. Incidentally both the head and the Tee give a buttress considerable stiffness in a lateral direction so that in general, few, if any, braces are required between the buttresses.

9. The relative slopes of both up-stream and down-stream faces of this type of dam result in a very favorable distribution of the principal stresses in the buttress walls. In a good design the maximum principal stress will be of nearly uniform intensity in horizontal sections over a large portion of the dam, and the second principal stress can be made compression throughout.

10. The round head buttress type of dam is well adapted to overflow by providing a reinforced concrete deck slab between the sides of the down-stream Tees.

11. This type of dam is well suited for construction in stages. The shape and cross section of the buttress head is practically the same for small and for large pressures, and an increase in the height of the dam can be secured very economically by the addition of an inclined column unit at the down-stream side.

12. The round head buttress type is best suited for dams of moderate and great height. Long spans are preferable to short spans on account of the fewer buttresses needed and the consequent greater concentration of mass in the fewer units. The composite types involving buttress units and slabs or arches will in many cases permit some additional economy.

13. A round head buttress dam requires materially less concrete than a gravity dam though considerably more than a thin section reinforced concrete structure. In cost it compares in many cases favorably with either type.

Readers are referred to the JOURNAL *for April 1933, for discussion which may develop. Such discussion should reach the Secretary by February 1, 1933.*

Lenora Street Viaduct

BY CURRAN CAVANAGH*

The Lenora Street Viaduct in Seattle, completed in June, 1930, at a cost of $36,000 was designed by means of the moment and shear diagrams given in Norman M. Steinman's paper in the Journal of the American Concrete Institute for January, 1930.

Some of the distinguishing features in reinforced concrete design were the use of concrete at a working stress of 1200 p. s. i.; continuous girder spans up to 64 ft. long freely supported on hinged joints; anchorage to take care of the grade and the tractive effort; the carrying of wind stresses to the end bents; and the use of flexible columns to relieve the bending stresses due to temperature movement.

When, in March, 1930, the Port of Seattle Commission signed the contract to build a new terminal to serve the Canadian Pacific Steamships and agreed to have it ready for the summer tourist traffic it became necessary to provide a suitable approach from the second floor waiting room to the streets on higher ground parallel to, but 400 ft. back from the waterfront.

A contemplated improvement plan for Railroad Avenue, which the viaduct spans, further complicated the locations of bents for they had to fit in with the existing as well as the future structures. This accounts for the odd length spans and the temporary timber portion adjacent to the dock.

A minimum time allowance, availability of materials, permanence, appearance and low initial cost and maintenance were the factors influencing the selection of concrete as the most suitable material for construction.

The girder type of construction was found most suitable for the long spans required. The first studies contemplated a rigid frame but it was soon discovered that unequal loading of the roadway caused moments in the columns which would require larger columns than

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FIG. 1—LENORA STREET VIADUCT

clearances or appearance would permit. To hold down column sizes it was also found advisable to use an anchorage for the assumed transverse force rather than to divide it between the bents. For a similar reason the wind stress was taken through the deck acting as a horizontal beam to the ends of the structure, at the hill end directly to the ground and at the waterfront end to a braced bent.

The diagrams proved very useful for all these studies in the relation of column to girder sizes.

The design of the roadway slab was based on a live load of the rear axles of two 15-ton trucks passing, 75 per cent of the weight being on the rear wheels, with 100 per cent impact on one rear wheel of each truck. This load was taken by a strip of slab 6 ft. wide at $\frac{Wl^2}{12}$.

For the girder design this same load in the center of the span was equivalent to a uniform live load of 117 lb. per sq. ft. and this load was used in the diagrams in working out the moments and shears in the girders as continuous beams freely supported.

Other design assumptions were:

Temperature coeficient, .0000055 for a range of 50 deg. In this connection it might be noted that the anchorage at one end made for a maximum temperature movement at Bent 6 and the compression in the upper 10 ft. of the pile foundation was computed as relieving the bending stresses in the columns.

F_s, 16,000 p. s. i.

F_c, maximum of 1200 p. s. i.

Shear, 60 p. s. i. for beams with no web reinforcement and without special anchorage of longitudinal steel; 90 p. s. i. with no web reinforcement, but with special anchorage of longitudinal steel; 180 p. s. i. with properly designed web reinforcement, but without special anchorage of longitudinal steel.

Wind load, 30 lb. per sq. ft. on a 9-ft. projection.

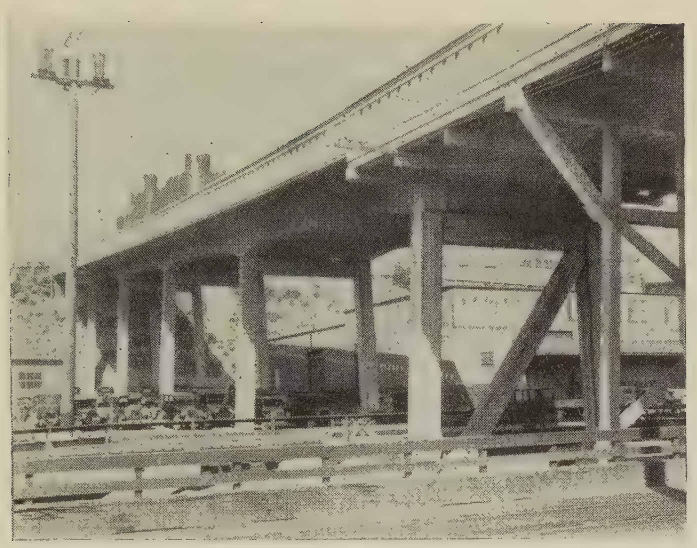

FIG. 2—JUNCTURE OF CONCRETE BENTS WITH TIMBER PORTION ADJACENT TO DOCK

Transverse force, 20 lb. per sq. ft. or 400 lb. per lineal ft.
Live load on walk, 80 lb. per sq. ft.
Bearing on bronze pin 5000 lb. per lin. in. × diam. of pin in inches.

The footings under bents 1, 2 and 3 were on hardpan, under bent 4 on untreated piles and at bents 5 and 6 on creosoted piles, the last two bents being subjected to the action of tidal waters. Provision was made at bent 6 for the connection of a future plate girder span when the street improvement is completed.

The inspection of the concrete for the viaduct was carried out under the following plan:

As the design strength was 3000 p. s. i. a mix of 1:1¾:2¾ containing 7 sacks of cement per cu. yd. was decided upon. The concrete was mixed at a commercial plant approximately two miles from the site of the work and was hauled to the job in 2-yd. batches by truck. The moisture content of the sand and gravel ran fairly uniform as the raw materials were loaded on barges and floated to the mixing plant where they were stored in bunkers, thus having ample time to dry out.

The aggregates were weighed and the water was measured for each batch. Thus uniform batches were easily obtained. Slump tests were made frequently and the water was varied accordingly. The following slumps were aimed at in the different parts of the structure: footings, 4 in.; columns, 5 to 5½ in.; beams and girders, 6 in.; deck, 5 to 6 in.

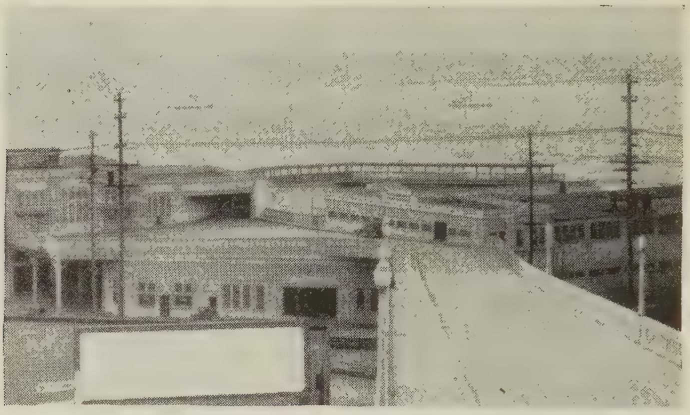

FIG. 3—DOCK END OF VIADUCT AND STEAMSHIP TERMINAL

The footings were poured three weeks in advance of the columns. The columns were poured continuously at the rate of 3 ft. per hr., several being poured at the same time. The slabs, girders and cross-beams were poured continuously, the pour lasting 28½ hr. and approximately 600 cu. yd. of concrete being placed.

Acknowledgment is made to Norman M. Steinman for the great help his paper afforded; to Jacobs and Ober, Seattle, for permission to use their hinge design; and to C. H. Eldridge of the City of Seattle Bridge Department for many valuable suggestions relating to design.

The viaduct was designed by the writer in the office of the Port of Seattle Commission under the direction of J. R. West, chief engineer.

Readers are referred to the JOURNAL *for April 1933, for discussion which may develop. Such discussion should reach the Secretary by February 1, 1933.*

THE HYDRATION OF TRICALCIUM ALUMINATE

BY W. D. FOSTER*

INTRODUCTION

A STUDY of the hydration of tricalcium aluminate is important as it is apparently one of the minerals responsible for the initial set of a portland cement. Rankin and Wright first proved its presence in portland cement by their phase rule studies on the $CaO\text{-}Al_2O_3\text{-}SiO_2$ system. (1)† More recently Brownmiller and Bogue have shown its presence in commercial portland cements by the use of X-rays. (2) Klein and Phillips (3) and later Wells (4) have studied the hydration of tricalcium aluminate and agree that the reaction as a whole is the simple addition of water to the anhydrous tricalcium aluminate as follows:

$$3CaO.Al_2O_3 + nH_2O = 3CaO.Al_2O_3.nH_2O$$

Thorvaldson (5) has studied the products of this reaction and reports four hydrates,

$3CaO.Al_2O_3.6H_2O$
$3CaO.Al_2O_3.8H_2O$
$3CaO.Al_2O_3.10.5H_2O$
$3CaO.Al_2O_3.12\ H_2O$

The hexahydrate is isometric and is formed at relatively higher temperatures than the others. The other three hydrates are hexagonal and for practical purposes indistinguishable.

The purpose of this paper is twofold; first, to study the character and the approximate rate of hydration of tricalcium aluminate in water and various salt solutions, principally calcium chloride and sulfate; second, to determine the rate of hydration of tricalcium aluminate in water alone and with calcium chloride and sulfate admixed. The character of hydration was observed microscopically and the products of hydration identified by their optical properties. Photomicrographs were taken frequently for illustration. For an accurate determination of the rate of hydration the Hubble (8) method was chosen. This will be explained in the appropriate section.

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†Figures in parenthesis are to references listed in the bibliography at the end of this paper.

THE CHARACTER OF HYDRATION OF TRICALCIUM ALUMINATE

When tricalcium aluminate is added to water, it passes into solution with dissociation, giving among other things, calcium ions. The calcium and alumina in solution then recombine as the tricalcium aluminate hydrate and this precipitates. (5)

Calcium ions already present in solution from a soluble calcium salt should retard the formation of calcium ions from the tricalcium aluminate and thus retard its hydration. Dilute solutions of all soluble calcium salts available, which included the calcium chloride, sulfate, acetate, bromide, chlorate, chromate, hydroxide, iodide, lactate, nitrate, nitrite and permanganate, were tested. They all retarded the hydration of the tricalcium aluminate.

The hydrate of tricalcium aluminate is amorphous at first and later crystallizes. In water this crystallization of the amorphous gel-like hydrate takes place almost immediately. Fig. 1 shows tricalcium aluminate in water for 10 minutes. There is a border of needle like crystals around the grains (in circle) and much amorphous (in triangle) and crystalline hydrated material (hexagonal plates in squares) a short distance out from the grains.

Dilute solutions of calcium salts, besides retarding hydration, strongly inhibit crystallization of the amorphous hydrated material. Fig. 2 shows tricalcium aluminate in a saturated gypsum solution for 10 minutes. The hydrate is forming slowly as globules of amorphous material on the surface of the grains (in circle). Similar conditions were observed when a saturated calcium hydroxide solution and a very dilute calcium chloride of approximately the same normality as the gypsum and calcium hydroxide solutions were used.

Two, 4 and 8 per cent $CaCl_2$ solutions acted similarly in inhibiting crystallization. The amount of hydrated material at 10 minutes, or the rate of hydration, increased with the concentration of the calcium chloride, until the rate of hydration in the 8 per cent solution was approximately the same as in water alone. A nearly saturated calcium chloride solution, about 37.5 per cent $CaCl_2$, acted more rapidly yet. The tricalcium aluminate was rapidly breaking down to an amorphous mass. At the end of one hour it was practically completely broken down to an amorphous homogeneous mass and hydration was complete.

Tricalcium aluminate was also hydrated in 2, 4, and 8 per cent $CaCl_2$ solutions saturated with gypsum and in similar calcium chloride solutions carrying gypsum suspended in them. No appreciable difference was noted between the effect of these and the ones without

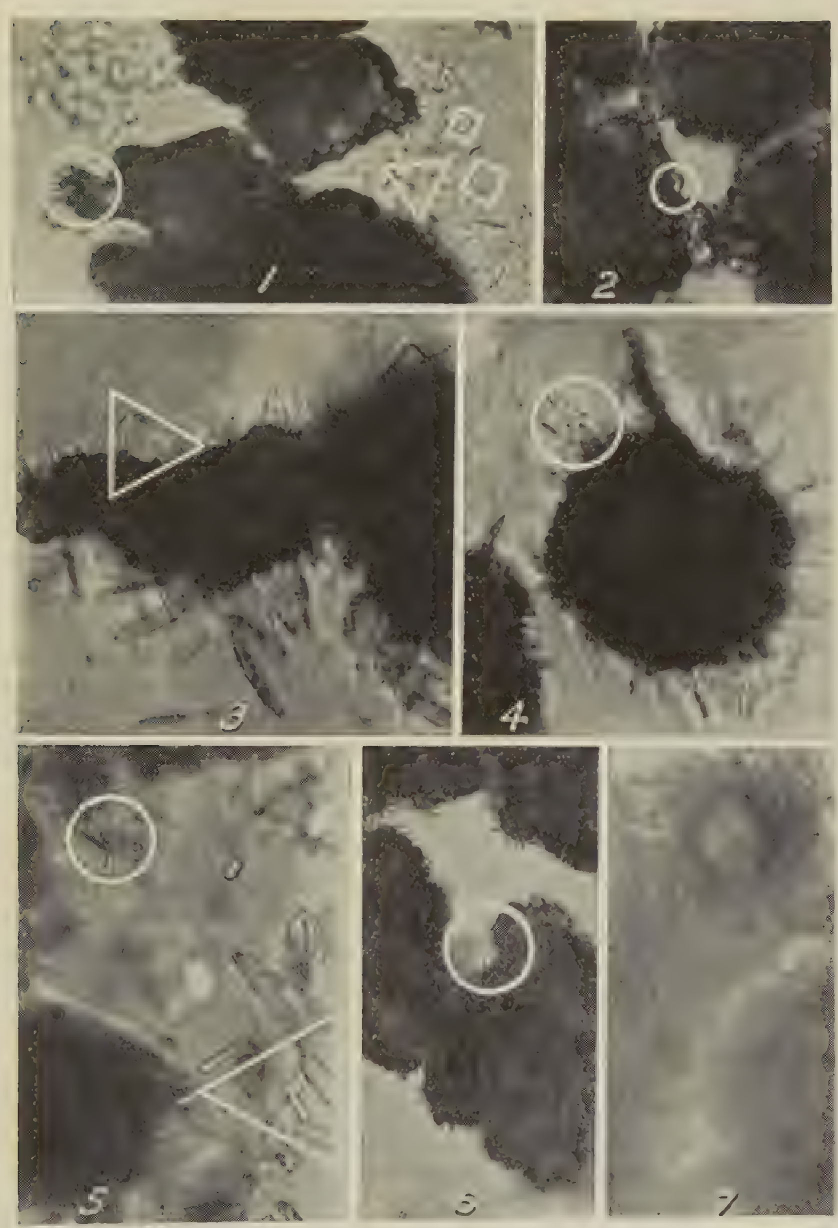

Fig. 1-7—(Magnification 359 diameters)

gypsum. Crystals of calcium sulfoaluminate had not appeared at 10 minutes.

When something is present to delay crystallization of the amorphous tricalcium aluminate hydrate, it frequently happens that the grains of tricalcium aluminate can be seen glued together by a gel forming between them. This gluing together will give a stronger bond than that resulting from the interlacing of crystals, and is more to be desired in the setting of portland cement than the interlacing crystal type of bond.

The previous discussion has been on the early hydration of tricalcium aluminate, illustrated by conditions at 10 minutes. What follows is on the later hydration, illustrated by conditions at 24 hr. when hydration is practically complete.

After 24 hr. in water tricalcium aluminate shows an abundance of crystalline hydrated material. (Fig. 3). Sometimes, instead of plates as shown in the figure, a thick border of fine needles was formed.

Tricalcium aluminate in a saturated solution of gypsum shows large ragged crystals of the hydrate; (Fig. 4) nothing resembling the sulfoaluminate is present. If an excess of gypsum is present, (Fig. 5) the calcium sulfoaluminate can be seen bordering the grains and in detached spherulitic clusters (in circles). The unevenly distributed broader and longer crystals (in triangle) are the excess gypsum. The use of excess gypsum has insured the conversion of the hydrated material to the calcium sulfoaluminate.

With either a saturated calcium hydroxide solution or a calcium chloride solution of approximately equivalent normality, the hydrate appears as fewer and much smaller crystals. These solutions apparently still inhibit crystal growth at 24 hr. Fig. 6, for the purposes of reproduction, illustrates this, though it is really a picture of the action of 4 per cent $CaCl_2$ solution saturated with gypsum. Nothing was seen to indicate the formation of a tetracalcium aluminate hydrate (4) with a solution of calcium hydroxide or an excess of calcium hydroxide present.

Tricalcium aluminate in 2 per cent $CaCl_2$ solution for 24 hr. is bordered with fine crystals, very similar to Fig. 6. In 4 per cent $CaCl_2$ solution, the hydrated material is amorphous at 24 hr. but soon crystallizes.

Tricalcium aluminate hydrated in 2, 4 and 8 per cent $CaCl_2$ solution saturated with gypsum is very similar to the calcium chloride solution (Fig. 6). In the 8 per cent $CaCl_2$ solution saturated with gypsum, the two salts, apparently working together, have caused a breaking down

of the tricalcium aluminate leaving well defined nuclei embedded in a mass of amorphous material. This had not appreciably crystallized at nine days.

When tricalcium aluminate is hydrated for 24 hr. in 2, 4 and 8 per cent $CaCl_2$ solutions carrying excess gypsum in suspension, it is broken down, apparently by the combined action of the two. With 2 per cent $CaCl_2$ solution and excess gypsum there are prominent nuclei and a small amount of amorphous material. With 8 per cent $CaCl_2$ solution plus excess gypsum the grains are practically completely broken down with almost no evidence of nuclei and with heavy amorphous hydrated material. The 4 per cent $CaCl_2$ solution with excess gypsum is between the extremes and is illustrated in Fig. 7. The amorphous material in these did not crystallize appreciably in several weeks of observation.

These experiments show calcium chloride and gypsum to have an additive effect in retarding the crystallization of the material resulting from the hydration of the tricalcium aluminate. The apparent greater effectiveness of calcium chloride than gypsum in inhibiting crystallization of the hydrated tricalcium aluminate may be due only to the greater solubility of the calcium chloride, as at the same concentration the action is nearly identical.

If calcium chloride is used integrally in portland cement, there will be approximately a 4 per cent $CaCl_2$ solution with excess gypsum. Tricalcium aluminate alone in such a solution would be broken down as in Fig. 7. Whether or not the same thing would happen in portland cement would have to be learned by further study.

RATE OF HYDRATION OF TRICALCIUM ALUMINATE

The rate of hydration of tricalcium aluminate has been determined by the Hubbell method (6), (7), (8). As this method is described in detail in the references given, it will here be summarized only. This method depends on a drop in index of refraction from the anhydrous mineral to the hydrated material. Anhydrous tricalcium aluminate has an index of refraction of 1.710, while the hydrated tricalcium aluminate ranges in index from 1.505 to 1.604, depending on which hydrate is formed. Some of the partially hydrated material is ground very fine, mixed with an oil whose index of refraction is 1.67, and observed under a microscope. The hydrated particles have an index less than this oil, and the unhydrated have a higher index. By focusing the microscope on a plane slightly below the particles, the hydrated ones will appear light and the unhydrated ones dark. A field is chosen at random, and the number of light and dark particles counted. If

enough particles are included, an accurate value of the per cent of hydration can easily be calculated.

There are two corrections necessary in order to obtain the value of the per cent of hydration from the count of the relative number of high and low index particles. Before hydration starts, there are already some low index particles present. These come from impurities in the tricalcium aluminate, such as a small amount of calcite; or from an added quantity of gypsum, which was used in two different proportions in these experiments. To correct for this, unhydrated samples are ground fine and a particle count made to determine the original percentage of low index material.

When tricalcium aluminate hydrates, its density drops. The values obtained after the correction for the original amount of low index material are really volume per cent. To correct for this the change in density must be known. As this whole correction is within or near to the limits of the experimental accuracy of this method, values taken from the literature for the density of tricalcium aluminate and its hydrated products are accurate enough (6) (9).

A further error, for which it is difficult to correct, arises from the difference of hardness of the tricalcium aluminate and its hydrated products. The tricalcium aluminate, being harder and tougher than the products of hydration, does not reduce to as fine particles as the hydrated material even on prolonged grinding. This will cause the determined value of the per cent of hydration to be at all times higher than the true per cent of hydration.

In this series of studies five identical trials were made up for each different set of conditions under which the tricalcium aluminate was hydrated. At each time at which hydration was determined, three of the five trials were sampled. These were dried, ground fine, and counted as above. Two different fields were counted on each, counting the high and low index particles in quadrants one and three in one, and quadrants two and four in the other. Counting was stopped in each quadrant when a total of about 75 particles had been counted. A total of about 40,000 particles were counted in order to draw the curves shown here.

Tricalcium aluminate was made according to the methods used by the Bureau of Standards (4). It was impossible to eliminate a trace of free lime by repeated firings, but this turned to calcite before any experiments were run. Sized particles were used for this work and for the microscopic work in the first part of the paper. These passed a 200-meshed screen but were held on a 300-mesh screen.

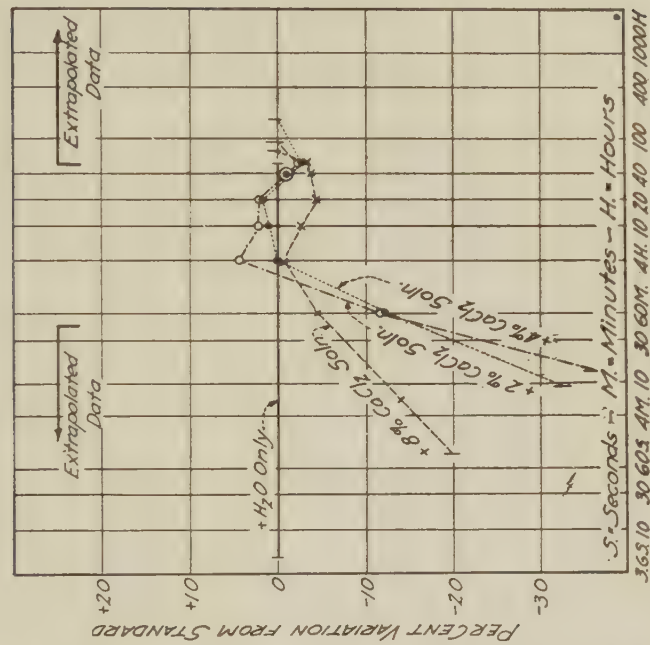

FIG. 9—VARIATION IN AMOUNT OF HYDRATION FROM STANDARD (WATER ONLY)

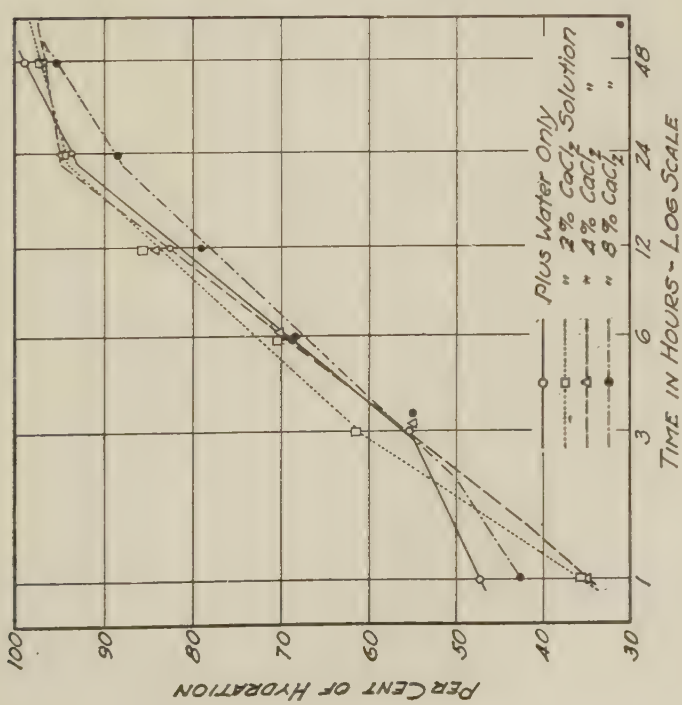

FIG. 8—RATE OF HYDRATION OF TRICALCIUM ALUMINATE

In all the trials, one gram of tricalcium aluminate was placed in a four inch test tube, one cubic centimeter of hydrating solution added, the two mixed, corked, and stored in a constant temperature oven at 35° C. This was a slight excess of hydrating solution, enough to keep the tricalcium aluminate submerged.

Tricalcium aluminate was hydrated under seven different conditions: in plain water; in 2, 4 and 8 per cent $CaCl_2$ solutions; in plain water with 2 and 10 per cent gypsum added; and in 4 per cent $CaCl_2$ solution with 2 per cent gypsum added. Samples were taken at 1, 3, 6, 12, 24 and 48 hr., dried, ground fine, and number of particles above and below 1.67 counted.

Fig. 8 shows the rate of hydration of tricalcium aluminate in plain water and in 2, 4 and 8 per cent $CaCl_2$ solutions. The time is plotted on a logarithmic scale which allows one or more straight lines, fitted to the points by the method of least squares, to be drawn to show rate of hydration, and incidentally more evenly distributes on the figure the times at which the amount of hydration was determined. Notice that the base line in this figure is at 30 per cent hydration. The symbols showing the amount of hydration at the various ages are the average of three determinations.

There is no significant difference between these curves except previous to 3 hr., where the amount of hydration in the various calcium chloride solutions is less than the amount in plain water. The retardation of hydration is roughly inversely proportionate to the concentration of calcium chloride.

To illustrate more easily this early retardation, Fig. 9 was drawn. The curve for tricalcium aluminate in plain water is shown as a straight horizontal line. At the various ages the plus or minus variation of the amount of hydration in the calcium chloride solutions from the amount in plain water is calculated and plotted. All the curves shown are extrapolated from the known data to cover the complete time of hydration of the tricalcium aluminate in each solution from 0 to 100 per cent hydration. The time is again plotted on a logarithmic scale. For convenience, the experimentally determined points are not all used; instead, points were interpolated at other ages. Such an interpolation from a curve whose position has been calculated by the method of least squares is fairly accurate.

From the curves on Fig. 9, the retarding effect that calcium chloride has on the hydration of tricalcium aluminate can be more easily seen. The curves for hydration in the calcium chloride solutions run much below the curve for the hydration in plain water at the early ages, and do not meet it until at about three or four hours. From here on to the

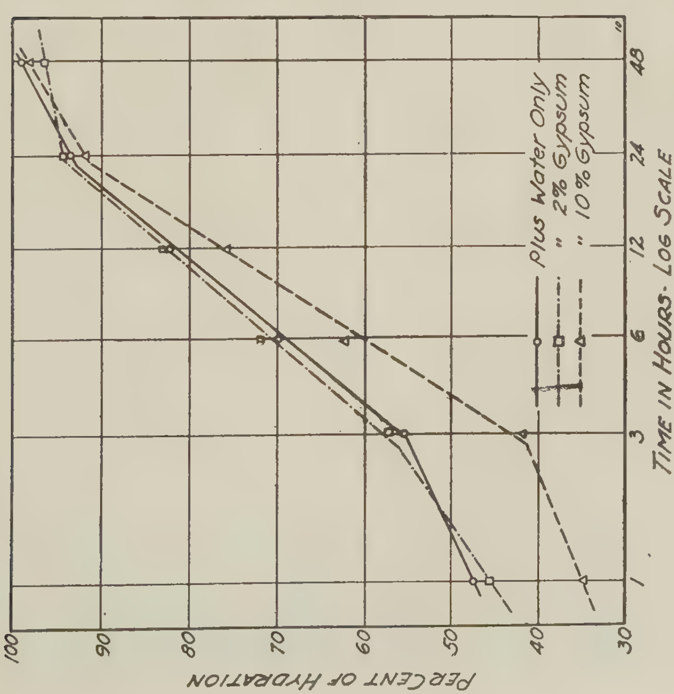

FIG. 10—RATE OF HYDRATION OF TRICALCIUM ALUMINATE

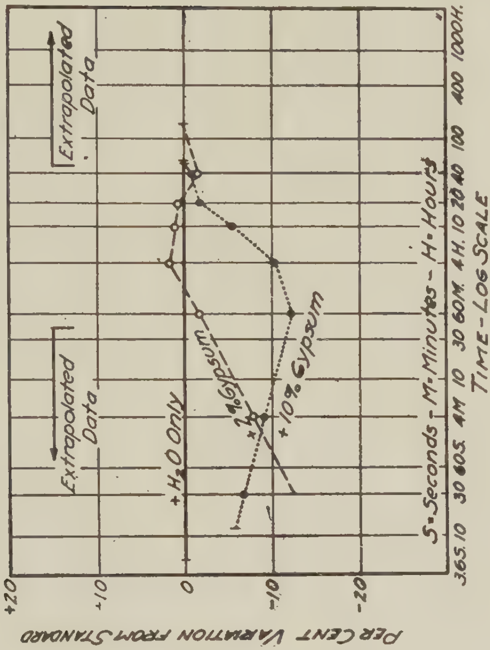

FIG. 11—VARIATION IN AMOUNT OF HYDRATION FROM STANDARD (WATER ONLY)

finish of hydration there is no significant difference. The amount of retardation at the early ages is apparently inversely proportionate to the concentration of the calcium chloride. With such large extrapolations there is a large degree of uncertainty, and while the 8 per cent $CaCl_2$ is probably a poorer retarder than the other two concentrations, quantitative comparison of the 2 and 4 per cent $CaCl_2$ solutions is not justified.

These curves seem to show, also, that calcium chloride is effective as a retarder for the first several hours, but after this its effectiveness practically ceases. This is probably due to the withdrawal of calcium chloride from the solution to form a calcium chloraluminate with the tricalcium aluminate.

Fig. 10 compares the rate of hydration of tricalcium aluminate in plain water with that of tricalcium aluminate to which 2 and 10 per cent gypsum has been added. About 2 per cent of gypsum is added to portland cement as a retarder, so a similar amount was used with one batch of tricalcium aluminate. As tricalcium aluminate at its maximum will be about 20 per cent of the portland cement, the amount of gypsum added is 10 per cent of this; so a second batch was used containing 10 per cent gypsum. Fig. 10 is drawn similarly to Fig. 8, and its previous description will apply.

There is no difference noted on this figure between tricalcium aluminate hydrated in plain water and tricalcium aluminate with 2 per cent gypsum added. The tricalcium aluminate carrying 10 per cent gypsum is about 12 per cent lower at one hour than the tricalcium aluminate without gypsum. This difference gradually disappears until at 24 hr. it is practically gone.

To show up the difference in the early hydration better, Fig. 11 is drawn on the same plan as Fig. 9. The tricalcium aluminate hydrating in plain water is again used as a standard, and the per cent variation of the tricalcium aluminate plus gypsum mixtures is calculated and plotted. All curves are extrapolated to cover the entire time from 0 to 100 per cent hydration. The time is plotted on a logarithmic scale as before.

From this figure it can be seen that the 2 per cent gypsum added to the tricalcium aluminate retards the hydration of the tricalcium aluminate at the start, but its effect disappears at about one hour; also the 10 per cent gypsum added to the tricalcium aluminate retards its hydration at the start and up to about 24 hr., as has already been noticed. Due to the large amount of extrapolation involved, quantitative comparison of the amount of retardation produced by the two different concentrations of gypsum is not possible. All that can be

said is that they are approximately equal. Here again the probable cause of the disappearance of the retarding action of the gypsum is the combination of the gypsum and the tricalcium aluminate to form a calcium sulfoaluminate.

Information determined in the same way as to the retarding effect of gypsum mixed with calcium chloride solution shows that, considering only a 4 per cent $CaCl_2$ solution and a 2 per cent admixture of gypsum, the best retarder is the 4 per cent $CaCl_2$ solution, the next best a mixture of the two, and the poorest the 2 per cent admixture of gypsum. In other words, the retarding action of the gypsum and calcium chloride is not additive in a mixture of the two; instead of this the retarding action of the mixture is near an average of the effect of the two salts taken separately.

Calcium chloride and gypsum both retard the hydration of tricalcium aluminate. The effect of the calcium chloride seems to be stronger during the early hydration, during a period corresponding roughly to the time taken for the initial set of the cement. After this its effect disappears. The 10 per cent of added gypsum apparently does not retard the hydration of the tricalcium aluminate as much at first, but its effect disappears more slowly, lasting to about 24 hr. This more rapid action of the calcium chloride, both as to greater early retarding and quicker disappearance of the retarding action, may be due entirely to the much greater solubility of the calcium chloride.

The retarding action of both the calcium chloride and the gypsum disappear before the hydration of the tricalcium aluminate is completed. This may mean that the calcium chloride and the gypsum disappear as such, combining with the tricalcium aluminate to form complex compounds. Unfortunately the fact of the disappearance of the retarding effects of the calcium chloride and gypsum was not realized in time to test the hydrating solutions to see if they had actually disappeared. In a previous paper it has been shown that in a solution in which some portland cement was hydrating, the concentration of calcium chloride dropped. (9) Also, in this series of tests, there is enough tricalcium aluminate present to combine with all the calcium chloride in the 2 and 4 per cent solutions, if they form the calcium chloraluminate reported in the literature, (5) $3CaO.Al_2O_3.CaCl_2.10H_2O$. It so happens that the 8 per cent $CaCl_2$ solution, in which the tricalcium aluminate might not be able to take up all the calcium chloride, shows a slight amount of retardation when this effect has disappeared in the other calcium chloride solutions. Whether this is more than a coincidence is not known at present. The tricalcium aluminate also would be able to combine with all the gypsum

used in the trials in this paper if it forms either of the calcium sulfoaluminates reported in the literature (10).

ACKNOWLEDGMENT

This paper is a joint report of the Calcium Chloride Association and the Ohio State University Engineering Experiment Station, resulting from their cooperative investigation of the action of calcium chloride on portland cement.

This work is under the direction of R. C. Sloane, professor of highway engineering and W. J. McCaughey, chairman of the department of mineralogy, Ohio State University, to whom the writer wishes to express his appreciation for interest shown and assistance rendered. Thanks are due to H. F. Clemmer, representative of the Calcium Chloride Association, for helpful suggestions.

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Readers are referred to the JOURNAL *for April 1933, for discussion which may develop. Such discussion should reach the Secretary by February 1, 1933.*

Sliding Form Work

Report of Committee 608

BY L. BOYD MERCER*, AUTHOR-CHAIRMAN

INTRODUCTION

The use of sliding forms in concrete construction is typically North American, and a comparatively new and high speed method. Its economy is in the fact that the cost of thin reinforced concrete walls depends largely upon the cost of the formwork, and hence upon material and labor used in making and erecting the forms. Thus the idea of building a single ring of formwork and sliding it up over the freshly placed concrete was evolved, and also the term "slip forms." An interesting stage in the development of sliding forms is recorded by Milo S. Ketchum in his book "Walls, Bins and Grain Elevators" where construction of bins for the Fisher Flouring Mills is described. The forms and platforms were pulled up with the aid of pulley blocks attached to posts passing through the platform floor. In 1885, a Texan named Carrico patented a sliding form raised by a windlass. A. E. Wynn in his book "Design and Construction of Formwork for Concrete Structures" describes a similar method for constructing grain bins.

The author wishes to thank the critic members of the committee (John H. Heindel, Leonard Construction Co., Chicago; W. R. Sproul, E. W. Sproul Construction Co., Chicago; Edmund Wilkes, Jr., Jones-Hettelsater Construction Co., Kansas City) for their cooperation and their very valuable constructive criticisms of the original draft of his report. In several instances the suggestions of Messrs. Heindel and Wilkes have been directly incorporated in the finished report, while differences of opinion are preserved in foot notes.†

The procedure now almost universally accepted for sliding form work is to carry the entire weight of the forms and working platform on vertical steel bars, or jack rods, set in the concrete wall. Considerable attention must be given to details in preparation and operation of

*Argentine representative for the Jones-Hettelsater Construction Co.

†A supplementary contribution by W. R. Sproul, with special reference to North American practice in using sliding forms for rectangular structures, will be published in the JOURNAL for February, 1933—EDITOR

this system because sliding forms act as a flexible machine and lack the rigidity of stationary forms. Any foreman or supervisor of sliding form work for the first time may expect many worries and will learn to appreciate the confidence or courage with which early experimenters must have been inspired.

There has been a surprisingly high degree of specialization in slip form construction in North America. In spite of the potential economies, very few organizations have made a success of this type of construction. This failure to capitalize sliding form methods is partly due to the inability of builders properly to plan their work and attend efficiently to the numerous details and partly due to lack of published information on the subject.

EQUIPMENT AND METHODS

Jacks. Of the several types of jacks for raising the forms, the one most commonly used is the hand-operated screw jack. which has 4 main parts: the rod-clamp, hollow screw, turn collar, and the nut attached to the yoke which supports the formwork. The rod-clamp is a clutch having two jaws which grip the jack rod, and at the top a collar within which the hollow screw can turn freely. This screw has a right hand square thread, and the jack rod passes up through the center. The screw is threaded through a nut rigidly attached to the formwork and the turn collar is attached to the top of the hollow screw. A typical screw jack, the Folwell-Sinks, is shown in Fig. 1. The screw is turned with the aid of a bar inserted into a hole in the turn collar, and since the nut cannot revolve, the yoke must move vertically. With one complete turn of the screw to the right, the forms will be lifted ¼ inch, and with a turn to the left, they will remain stationary, being held up by the adjacent rods so the jack itself must climb the rod. This latter movement is of course necessary whenever the lower cross member of the yoke reaches the level of the turn collar.

This type of jack may be operated by line shaft power, a procedure developed by the Barnett and Record Co. Then there are various patented jacks such as the pump jack used by the MacDonald Engineering Co. A recent U. S. patent provides a method for raising the forms by hydraulic power. The hand operated screw jack at present is the safest and surest method for raising the forms.

Jack Rods. The jack rods should be circular bars 1 in. in diameter, and for use with screw jacks may be of mild soft steel although a harder or higher carbon steel is generally advisable. The jaws of a jack will bite better into a soft than into a hard steel but a soft steel bar is more likely to bend and break out of the wall below the forms.

Hence the hardest grade of steel practicable for each type of jack should be determined by the user. Jack rods should always be placed in the center line of the wall. The distance between them is dependent upon the advisable load for each jack and upon the economy in bracing between the yokes, too many jack rods and yokes being unnecessarily expensive, and too few, making it difficult to turn the jacks and expensive to make the form sufficiently rigid. Spacing between jack rods which depends upon the bracing of the forms,will be influenced by the shape and section of the bins and in outside walls

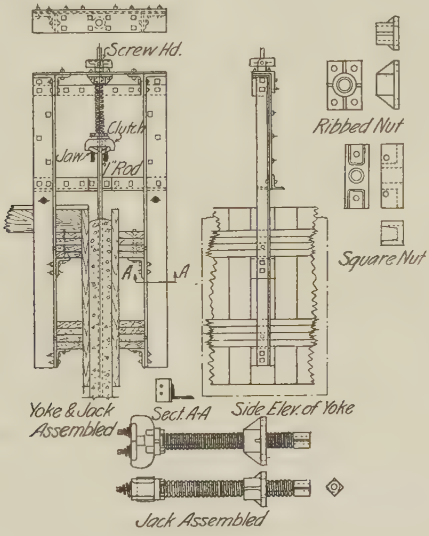


FIG. 1—THE FOLWELL-SINKS JACK AND FORM

of circular bins, between bin junctions, may be up to 10 ft. Straight, well braced, heavily constructed forms will naturally span farther than curved, bent, lightly built forms. Weak points in the jack layout require special attention and every effort should be made to eliminate them. Corners formed at the junctions of the outside bin walls are difficult to brace and hence the jack rods at these points should be close together. Fig. 2 (a) shows the advisable spacing and positions of the jack rods for circular bins, and Fig. 2 (b) shows their position for rectangular bins. Consideration must also be given when placing the rods, to convenience in wheeling and placing the concrete. The experienced supervisor is generally the best person for determin-

ing the advisable positions for the jack rods. In North American practice the jack rods in bin wall construction generally involve 10 to 15 lb. of steel per cu. yd. of wall concrete. This statement is furnished as a check on the designers' work rather than as a rule because every job is different and must be studied separately.

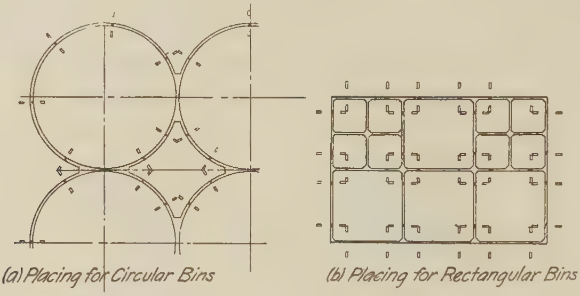


FIG. 2—POSITIONS FOR JACK RODS

The joining of jack rods is a simple matter but requires careful attention. When the jack reaches the top of the rod, it should be backed up clear, and a 4 to 6-in. length of 1-in. pipe sleeve forced half its length over the top of the rod. Denting the center of the pipe sleeve ensures that it will not pass too far over the rod. The new jack rod, passed through the hollow screw and inserted into the projecting half of the pipe sleeve, is gripped by the jack and lifting of the forms may proceed. For convenience the jack rods should be from 12 to 15 ft. long. R. F. Egelhoff—cf. *Engineering News-Record*, November, 1925—stated "various lengths of jack rods must be used at starting" but this is not necessary. The joint as described will be as strong and rigid as the bar itself and supervision can be simplified by extending all the rods at about the same time. Joining of all rods will never be required at exactly the same time, because the jack clamps will not be at exactly the same level. Thus the rods may all be the same length, and this length should be exactly divisible into the height of the bin wall from the top of bin bottom slab to the bottom of the slab over the bins. Mr. Heindel points out that although the above statement is correct with regard to small jobs, experience in larger work agrees with R. F. Egelhoff.* At the top of the bin wall it is necessary to add

*While Mr. Mercer is undoubtedly correct with respect to a small job with comparatively few jacks, however, on a fair sized job with 400 or 600 jacks, it seems obvious that it is far simpler to have the jack rods terminating at different levels. This need not be an indiscriminate arrangement but by starting the bottom jack rods in three lengths and having the balance of the rods all one length, the jack rod changes will come at regular points in the movement of the forms and create less distrubance in the progress of the work due to the fact that only one third of the total need be changed at one time. For instance, the bottom rods could be 12, 17 and 22 ft. and the balance 15 ft. with special lengths at the top. Thereafter, there would be one third of the jack rods to be changed each 5 ft. in height which can be done with a minimum crew with no confusion.—J. H. HEINDEL

short lengths, about 3 ft., of jack rod because the level of the top of the form is below the jack. These short lengths may be transported from one job to another to save cutting or bending, and perhaps wasting the tops of the jack rods.

Formwork. Sliding formwork construction may be considered in 3 sections—yokes, walings and sheeting. The yokes have two primary functions—to transmit the weight of the forms to the nut on the jack, and to prevent the wall forms from spreading. They may be either of steel as shown in Fig. 1 and 3 or of timber as shown in Fig. 4 and 5. Steel may prove economical when the yokes can be used many times within a small area, but timber is customary even with large and busy construction companies. Steel yokes are less likely to spring and rack than timber yokes and hence may prove suitable for the outside walls of circular bins and for straight walls. Builders must consider their own preferences and the volume and location of their work when choosing between timber and steel. Where timber is used, members of two or more plies, as shown in Fig. 5, generally will be found preferable to solid members, as shown in Fig. 4, because both strength and lightness are important. With steel yokes, the upright members carry the weight of the walings and hence the forms, as shown in Fig 1 and 3. Sometimes timber yokes are made to follow the same procedure, as in Fig. 4(a), but this is not good practice.

Vertical loads should be transmitted from the walings directly to the top member of the yoke or preferably to the nut on the jack. This can be done very conveniently by using two ¾-in. dia. steel lifting rods threaded at each end and secured to both the nut on the jack and the bottom waling as shown in Fig. 4(b) and 5. The upper waling is then supported from the bottom waling by bracing or filler timbers. These walings should also be made of several plies of timber. Where cutting of separate plies is to be done at the mill or by the timber merchant, the type of order-form shown in Fig. 6 may be used very conveniently. The separate plies should have their joints broken or staggered. R. F. Egelhoff suggests the use of solid walings, stating: "A certain amount of flexibility in the forms is produced by having the joints in the wales come directly under a jack clamp. A rigid form will rack and bind when it is jacked." When using several plies of timber in the walings, it will not be necessary to worry about the excessive rigidity, and it will prove much simpler when erecting the forms, to set the yokes without reference to these joints.

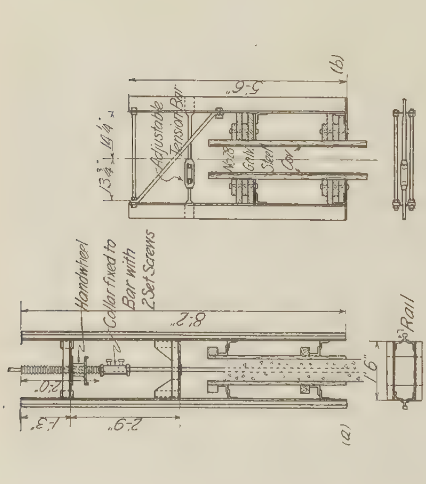


FIG. 3—(TOP, LEFT) TYPES OF SLIDING FORMS WITH STEEL YOKES

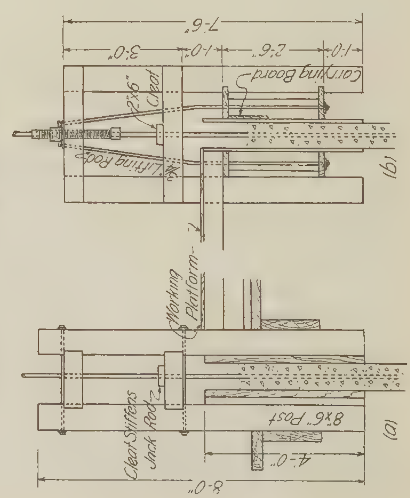


FIG. 4—(LEFT) TYPES OF SLIDING FORMS WITH TIMBER YOKES

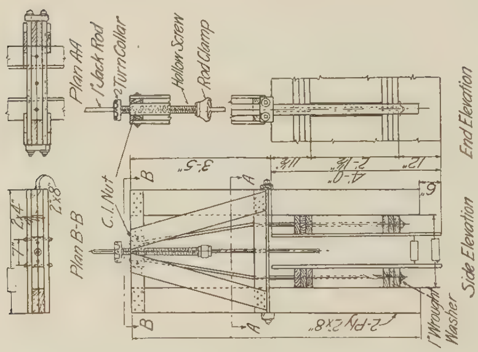


FIG. 5—TYPE OF SLIDING FORMS AND YOKE ADVISED IN TEXT

When the yokes are to be reused several times, it may be preferable to bolt the sections together rather than to use spikes or nails.

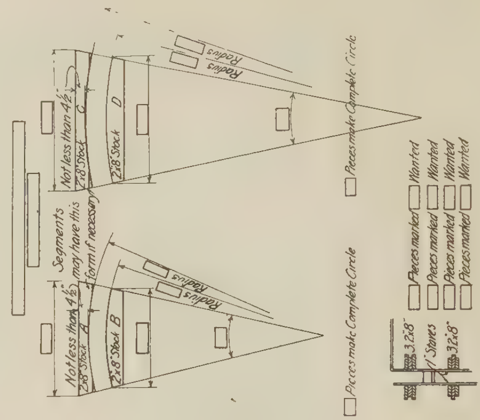


FIG. 6—(BELOW) METHOD FOR ORDERING THE CUTTING OF SWEEPS

Experience has shown that, for North American conditions, the most economical forms have the sheeting or wall-lagging 4 ft. high and made of dressed timber, one inch thick. For the finest workmanship, and where the lagging is to be used more than once, it may be specified as edge grain flooring in order to eliminate splintering as the forms slide. The staves or individual pieces should be 3 to 4 in. wide, depending on the diameter of the bin and hence the curvature required. Sheeting for the outside of the wall should be vertical, and the inside sheeting should be distant from it slightly less than the requisite wall thickness at the top and an additional ⅛ to ⅜ in. at the bottom. This slope of the inner form is necessary so the form will lift clear of the set concrete as it is jacked up. The concrete in the upper third or half of the form will be in a plastic condition and the wall will take its thickness from the level at which the concrete ceases to be plastic. The part of the forms below this level serves to prevent twisting and to float the surface. Hence the forms should be constructed slightly less than the wall thickness apart at the top and slightly more at the bottom. The amount of batter is a question upon which many successful supervisors differ. The working faces of the sheeting should be smooth, projecting edges of timber being planed down and the nail heads countersunk. Covering the faces with a sheeting of thin steel is a mistake as the joints and nail heads which cannot be countersunk tend to drag the concrete and do more damage than the softer irregularities of the timber. It is generally advisable to give the timber sheeting a coat of oil before starting. This is frequently done by dipping the staves in a tank of oil. Once the concreting is commenced, any alterations to the working surface of the forms are necessarily expensive.

Lifting the concrete. The shape of the connections between the bins is controlled by practical considerations rather than by design. As the forms are jacked up they tend to lift the recently placed concrete and the extent of this tendency depends upon the ratio of the volume of concrete in a section of the wall form to the area of that section. It is difficult, for this reason, to construct thin walls with sliding forms. The minimum thickness generally is accepted as 4 to 4½ in. and this is less than the advisable minimum where weatherproof walls are required. Where the concrete is molded into an acute angle, the ratio of volume of concrete to the area of the working faces forming the angle will be insufficient and the concrete will be lifted. There is even a danger of dragging the concrete when the inside forms make a right angle, and hence it is usual to have 4 by 4-in. fillets at all such points. The rein-

forcement, concrete mixture, and speed of operation, must also be adjusted to assist in overcoming the tendency to lift the concrete.

Platform. A necessary working platform over which concrete can be wheeled, must be, for convenience, level with the top of the molds, the usual procedure being to support it from the top inside walings. For economy, the platform should be designed also to serve as the formwork for the underside of the slab over the bins. The decking for the slab over the bins must withstand the combined loads from the wet concrete in that slab and from the concrete wheeling and tamping. Moreover, this decking must be supported by the wall forms, since it is impractical to support the formwork for the top slab with vertical timbers extending from the bottom of the bin. This slab must generally be designed to carry certain machinery and live loads such as conveyors and throw-off carriages. For the usual size of bin, 20 ft. diameter, these slabs can be constructed more economically if they have beams, than as slabs supported around the outside and reinforced in two directions. Some builders are misled by this conclusion and build the formwork for the concrete beams into the platform. It will be noted from Fig. 7, that the beam forms must be temporarily closed over for the wheeling of concrete, and also, when the top is reached, the concreting is generally stopped at the underside of the beams while the ends are cut through and the beams reinforced.

The most practical method for bins of about 20 ft. dia. is to use steel beams. Each case must be examined separately—for instance with very large circular bins, the steel beams would be so heavy and would add so much to the weight and cost of the forms as to be impracticable.

The concrete slab over the bin will rest on these steel beams and thus the tops of the beams must be level with the top of the timber decking. This necessitates using U-bolts to support a bearer, which in turn supports the joists under the lagging. Fig. 8 shows a method for cutting the ends of the beams and connecting angle bearers so that they may be supported by the top waling. It also shows the location of holes for U-bolts. The requisite dimensions for the steel beams must be determined for each case, and the depth of the timber joists will be the thickness of the decking, say 1 in., less than the depth of the steel beam. One end of each timber joist will be supported by the central bearer, and the other end can be packed up from, or notched into, the upper waling. Fig. 9 shows the plan of such a platform. These steel beams can sometimes be located beneath the walls of the houses over the bins. Care must be taken, however, in the design of these combinations of steel and concrete, because although a steel

beam is sufficiently strong, it will be more flexible and may cause cracks in the concrete. The judicious adoption of thin, properly reinforced concrete slabs and suitable steel beams will give satisfactory results.

FIG. 7—LA CONTINENTAL ELEVATOR IN BUENOS AIRES
Complicated work arising from the inclusion of concrete beam formwork in a sliding form platform.

Swelling of Forms. Care must always be taken in construction of timber forms to provide for expansion of dry timber when it comes into contact with wet concrete. The extent of this expansion will depend upon the amount of moisture already in the timber. This varies with the weather and the facilities for seasoning and can only be determined

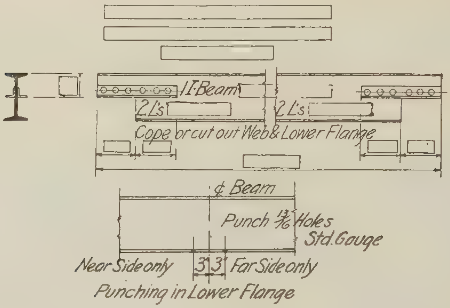


FIG. 8—DETAIL OF STEEL BEAM UNDER ROOF SLAB

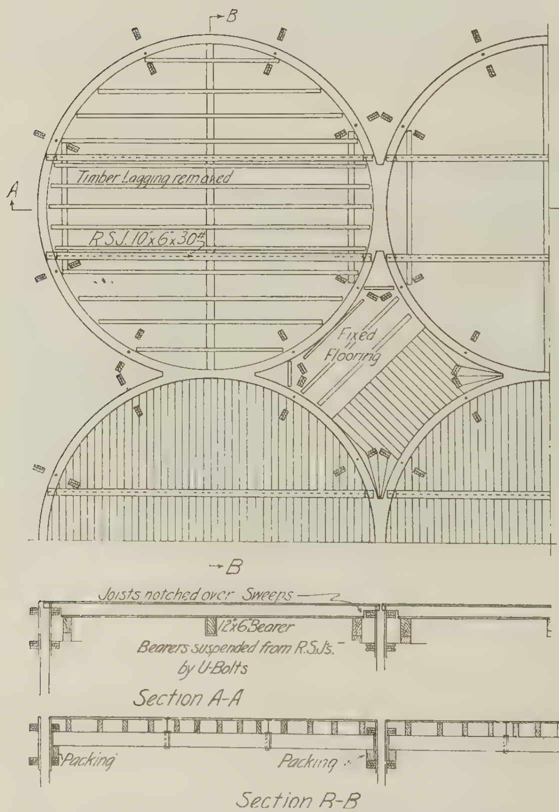


FIG. 9—WORKING PLATFORM FOR SLAB OVER BINS

by experience. In stationary formwork, the forms may be adjusted between the successive lifts, but with sliding forms, it is advisable to take the necessary precautions before starting.

Numerous methods for counteracting the effect of expansion of the timber upon the shape and size of the molds include: the contact ends of the walings may be painted with crude oil to prevent them absorbing moisture; the staves, or separate pieces of the sheeting, may have bevelled edges or may be set with V-joints, so that the edge of the timber will crush when the staves expand; the staves may be coated with oil or boiled in a waterproofing solution. Methods involving the formation of pilasters by formwork in which the panel lagging can slide over the board forming the face of the pilaster, should not be adopted. These methods depend for success upon the swelling of the forms decreasing the width of the pilaster, and even the casual observer cannot fail to notice such a serious defect in the appearance. The effect of the expansion of the forms for circular bins is generally most noticeable where the formwork for two bins comes together, and is connected by the small cross form. The pressure due to swelling which tends to bend this cross form, can be relieved by setting tapered staves in the sheeting at either side, and withdrawing them as the pressure becomes excessive.

The most simple and direct method for dealing with the swelling of forms is to set the staves a small distance apart and then keep them soaked with water for a couple of days before concreting. Tongued and grooved timber is advantageous for the staves because it gives a closed and concrete-tight mold even though the tongue is not tight in the groove. At first it may be necessary to remove a stave and replace it by a narrower one, or to cut a waling, but the builder will soon learn the distance which the joints should be kept open while the timber is dry, in order to give close joints and the requisite dimensions when it is wet.

Erection of the Forms. The erection of the forms and the preparations for the first fill are carried out in a manner similar to that adopted where stationary forms are used. It is more essential with sliding forms, however, to have them start exactly level, and hence the slab upon which they are to be set must also be level. Fig. 10(a) shows not only an amusing effort at beautification but also the result of stupidity in design. It was not discovered until the sliding forms were ready for erection that there was nothing upon which the outer forms could rest. This resulted in a waste of time and money while the little ledge was being stuck on. Fig. 10(b) shows another view of this work when the sliding forms were in operation.

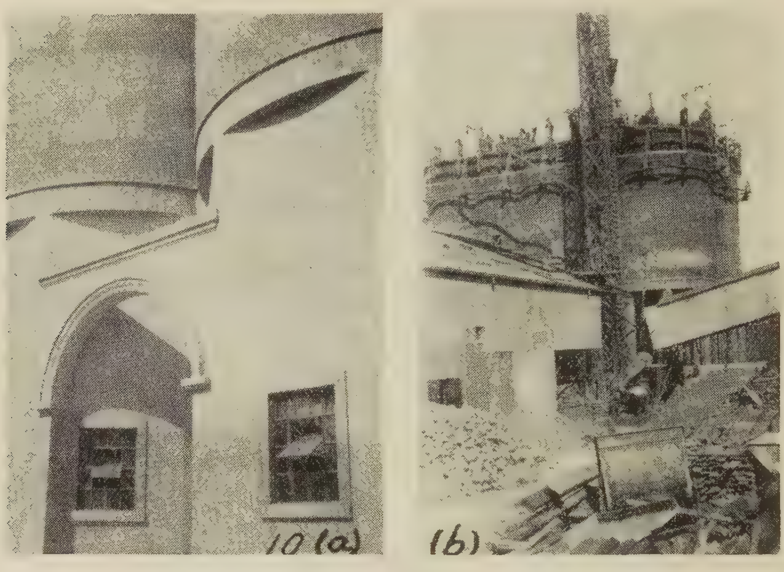
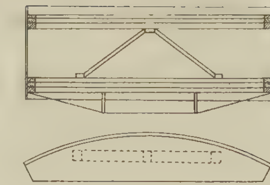


FIG. 10—AN EXAMPLE OF CARELESS DESIGN IN THE OMISSION OF A BASE FOR THE WALL FORMS

As with the stationary forms, after the outside molds have been set in position, the horizontal reinforcement may be placed up to the level of the top of these molds. This reinforcement can be tied to dowels, which should be projecting from the slab, and which will then assist in holding down the concrete of the first fill when the jacking-up commences. The inside molds can then be set in position and the walings bored to take the lifting-rods—thus the formwork can be supported from the jacks. The sheeting and walings may be connected and set-up in units in the method frequently adopted for stationary forms, or as in Fig. 11, which shows form erection for the new Buenos Aires Great Southern Railway Elevator at Ingeniero White. Incidentally, Fig. 11 shows a procedure of erecting the molds and reinforcing the wall different from that suggested above. It is customary in North America to follow this latter method and avoid the use of dowels because they interfere with placing the forms. The concrete is then placed in the forms in layers as deep as the vertical spacing of the bars. This method has the advantage of keeping the steel clear and free from a loose coating of half dried mortar which is several hours old. Another scheme when both the inside and outside molds are completed before the enclosed section of wall is reinforced, is to tie the horizontal bars to verticals and lower them into place. The lowering generally pro-



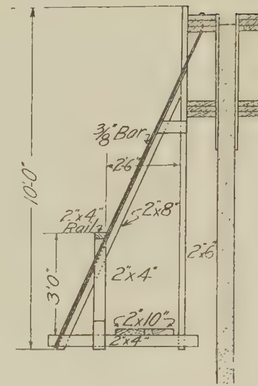


Fig. 11—(Top, Left) Construction of wall forms for the Buenos Aires Great Southern Railway elevator at Ingeniero White, Argentina

Fig. 12—(Above) Bracing between yokes

Fig 13—(Left) Detail of bracing between yokes

Fig. 14—(Right, Above) Scaffold for finishing sliding form walls

ceeds bar by bar as each horizontal ring is wired to the verticals. The walings are primarily designed to prevent the forms from bulging in a horizontal direction, but to prevent them from sagging between the yokes, they should be braced. When the spacing of the yokes is small, this bracing may be merely a carrying board attached to the sheeting and the underside of the walings, as shown in Fig. 4(b). For a spacing of, say 10 ft., however, the more definite bracing shown in Fig. 12 should be adopted. Frames with lifting rods, are sometimes placed between yokes to hold the walings. These frames (termed false yokes because they occur between the jack rods and must transmit their loads through the main yokes) are commonly used to support the forms for the internal junctions of bins or wherever the walings on one side are more easily supported than those on the other side. Hence false yokes are generally constructed to transmit their load through the walings to the nearest main yokes on the side which is most easily supported. Fig. 13 shows a type of sliding frame which is sometimes used for this purpose. The construction of the platform should complete the formwork and the jack rods may then be inserted through the jacks. These rods must be forced down so that they bear evenly on the concrete slab, and then the jacks should be turned enough to take the weight of the forms. The rigidity and balance of the entire framework can then be examined and any necessary adjustments made before starting to concrete.

Cost of Construction and Erection of Forms. The cost of preparing the wall molds will be somewhat higher than for stationary forms, but the greater part of the first cost of sliding forms is in setting-up, bracing and preparing the platform. Sliding forms for a simple 20-ft. diameter bin can be constructed with, as a general average, 20 "man-days;" a "man-day" representing the amount of work that can be performed by the average trained man for that work, in a working day. The total area of the wall is 515 sq. ft. and requires 15 "man-days" or costs approximately 3 "man-days" per 100 sq. ft. of finished surface. The area of the platform is approximately 330 sq. ft. and requires 5 "man-days" or costs approximately 1½ "man-days" per 100 sq. ft. of surface. These figures are general and approximate, being given for comparative purposes only.

Initial fill. The forms should be filled slowly at the beginning and jacking commenced as soon as the concrete has hardened sufficiently. The forms which will have to stand a greater pressure of green concrete at the beginning than at any subsequent period, are frequently protected from bursting during this period by temporarily bracing the

part below the lower waling from off the bin bottom slab and are prevented from collapsing at the top by the use of spreaders. These precautions may be ignored if the forms are very carefully designed and where the operators have had considerable experience in this class of work.

Much care will be necessary in timing the commencement of jacking so the concrete neither slumps away under the molds, nor grips them and is dragged upwards. The time for starting to jack depends upon the weather conditions, and the rate of hardening of the concrete, and hence can only be determined by experience. It is not usual to wait until the forms have been completely filled before starting to jack. This is because the concrete mixer will not need to operate at full capacity to supply the concrete required by the rise of the forms during the first few hours. The starting speed will always be slow but the builder who does this work for the first time will feel sufficiently relieved to have the forms move.

Wall Finish. The working faces of the molds tend to drag up the green concrete, and so it is necessary to keep the wall patched and rubbed down as soon after exposure as possible. The outside of the bins must receive this attention for the purposes of improvement to appearance and protection against weather.

The inside surfaces of bin walls do not require the good appearance demanded of the exterior or exposed surfaces although they must of course be sufficiently smooth to prevent the particles of grain from lodging or weevils from breeding. In working houses or wherever the interior areas are exposed, however, these surfaces must be treated the same as the outside. Hence it is usual to construct a continuous scaffold for this finishing work around the outside of the bins or building and to suspend interior scaffolds wherever the interior areas are exposed. These scaffolds may be supported by bracket hangers attached to the walings and may be of the type shown in Fig. 14. Fig. 14(a) shows a similar type of scaffold but a hemp rope is used for the hand-rail. These figures show sliding form work in Europe and by a European company. Fig. 15 shows a type of scaffold so designed that the finisher, as he moves around the platform, does not have to bend under diagonal members shown in the other type. Fig. 15(a) shows that the wall was patched but not rubbed-down evenly as the work progressed and hence necessitated washing-down as shown in Fig. 15(b). The excellent surface which can be obtained on the walls by rubbing them down correctly as the forms rise is shown in Fig. 16.

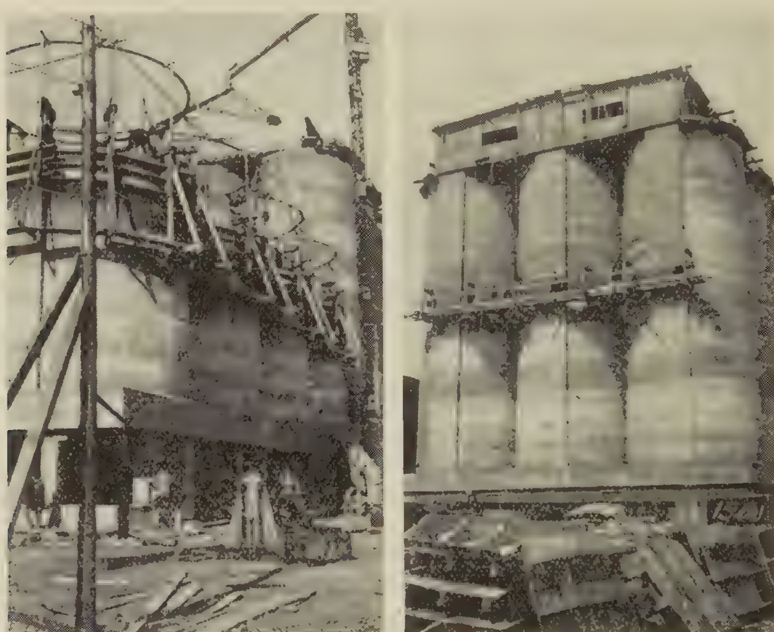


FIG. 15—WALL FINISHING AT ANTON JURGEN'S MARGARINEFABRIKIEN AT SPYCK ON THE RHINE, HOLLAND
(Reproduced by permission of Messrs. Christiani and Nielsen)

FIG. 16—CONSTRUCTION OF MAIN STORAGE, B. A. G. S. RY. ELEVATOR AT INGENIERO WHITE, ARGENTINA

Time and expense can thus be saved by finishing the wall as soon as it is exposed. This can be very simply performed by a wooden float to rub a film of cement and water into the surface of the green concrete. The inferiority of the final surface of this concrete as compared with that from stationary forms and the necessity for preventing the surface from drying too rapidly are discussed later.*

Raising Forms from Concrete. The forms leave the concrete because of the slope of the inside sheeting. Forms 4 ft. high should be clear of the wall when the top of the form is 12 in. above the top of the concrete. This means that a narrow space, say 1/16 in., can be seen between the form and the wall and thus shows that the form is not taking any pressure from the concrete. Sliding forms must be jacked clear in this manner whenever any unforeseen delays occur. These stops in the concrete placing will cause horizontal joints in the walls and these joints will show if the work is delayed sufficiently long for the top of one layer of concrete to harden before the next one is placed.

With careful organization and supervision it is possible in warm weather to concrete as much as a 20-ft. height of wall in 24 hrs. The concrete will then be free from the restraining influence of the forms within 1¼ hr. of the time of placing but will still be sheltered by the forms for a further 4½ hr. Nevertheless the concrete is seldom subjected to these severe conditions because the above stated speed of 20 ft. in 24 hr. is seldom attained. The average speed is generally nearer 10 than 20 ft. and the contractor should remember that breaking speed records is seldom economical or conducive to good workmanship. The above figures, however, demonstrate the possibilities of sliding form work.

Artificial Lighting. Since operations involved in actual construction of sliding form buildings necessitate night work, provision must always be made for artificial lighting of the platform. Recent developments in electric lamps and lighting methods make the old statement that outdoor night work was less efficient than day work, no longer true. The cost of the electrical installations and power consumption generally will be well repaid in the efficiency of the work. As a general rule the platform may be lighted with 150 watt lamps so arranged that each lamp serves approximately 75 sq. ft. of platform and that the concrete

*In the United States the finisher's scaffolds are commonly built with open-type hangers that allow free passage along the wall and permit inspection of every part. At this stage of the work it is not too late to repair and make weatherproof any imperfect portion of the wall. This is not true of stationary form work where forms are left on until the concrete is hard. The matter of the comparatively quality of sliding form concrete as against that produced with stationary forms is one in which this reviewer differs from the author, as his observation has led him to believe that with the proper workmanship the former is superior in texture and solidity to the latter for equal thickness of walls.—EDMUND WILKES, JR.

can be seen in the forms at all times. Whether the lines of lights follow the circles of bin walls or some other arrangement, must be determined for each specific case. Fig. 17 shows the illumination of the B. A. G. S. Ry. elevator sliding forms.

Organization of Labor. Successful operation of sliding forms involves the accurate and rapid performance of uniform work and hence the organization of the labor is a very important matter. There are 4 main operations to be considered, (1) handling materials and transfer of concrete from the mixer to the wall forms, (2) bending and placing reinforcement, (3) jacking or raising forms and (4) rubbing down exposed concrete. The two principal schemes for arranging the four operations are to make all of them continuous and simultaneous, or to subdivide them into two sections: concrete and reinforcement work and

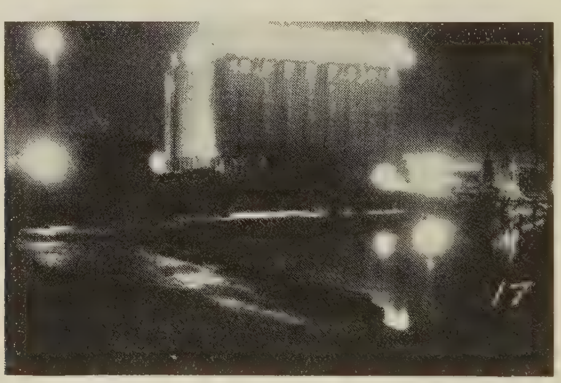


Fig. 17—Artificial lighting on B. A. G. S. Ry. elevator job

the jacking and finishing. Maximum speed for the completion of the walls demands continuous work, but does not necessitate that all the operations be simultaneous. The first scheme gives this maximum speed for a minimum capacity of concrete plant because the plant then can be theoretically designed for 24 hr. operation per day. The labor for this procedure generally will be engaged in three 8-hr. shifts for which there is often considerable difficulty in procuring sufficient men and skilled supervisors. Careful supervision is necessary, especially with the placing of the reinforcement, since the continuous placing of concrete and jacking very quickly cover the bars. This method, however, is ideal for working in warm weather, because it prevents the formation of those unsightly joints between the layers of concrete. The method can naturally give the maximum speed for the conditions of cement and weather.

There are numerous schemes for subdividing the labor operations into sections to conform with special local conditions of labor or the hardening rate of the concrete. When speed is of no importance, concrete and reinforcement work may proceed with one 8-hr. shift and the jacking and wall finishing in a second and later shift. The concrete workers then leave the wall filled up to the top and the jacking continues for a further 4 hr. or until the forms are clear of the concrete. Concrete may be left for 16 hr. in cold weather without hardening and hence the scheme for operation at slow speed with 12 hr. jacking then may be adopted. It is difficult to average more than 5 ft. per day by this method but a speed of 10 ft. per day can be obtained easily by having two 8-hr. shifts of concreting, and jacking for the entire 24 hr.

A modification of the above schemes can be made by concreting the walls in sections, 8 in. deep. When concrete work stops, the forms are jacked up a further 8 in. and another layer of reinforcement is placed. This necessitates designing the reinforcement with fixed vertical spacing which is wasteful of steel, but very greatly assists supervision of placing of bars. The scheme requires careful consideration in the timing of the shifts for day and night work, and in deciding the capacity for the concrete plant, but it greatly simplifies the supervision of the actual details of workmanship. The work by this method is continuous, and so the maximum speed can be reached without having all the operations simultaneous. Adjustments then can be made for night work and for cold weather. A further modification of this scheme is to continue jacking the forms slowly and continuously, placing the concrete in thin layers, each one being applied before the previous one has hardened. The thickness of these layers should conform with the spacing of horizontal rings of reinforcement. The organization of the labor for sliding form work requires considerable experience and should be very carefully planned in advance of the work.

Jacking and Leveling. The forms must be jacked up evenly and kept level, otherwise the walls cannot be vertical. Hence the jacking crew must be carefully organized and supervised. Each member of the crew should have a definite group of jacks allotted to him and must proceed around them in a fixed direction giving each one a half or 180-deg. turn and thus raising the forms ⅛ in. at a time. Where the men are not only unskilled at such work but are also unintelligent, the foreman of the crew must stand where he can see them all. He may then signal with a whistle when the crew is to start upon a round of jacking. The peons of South America, for instance, require super-

vision in this manner, and when the forms are rising 4 ft. in 8 hr., one man is incapable of attending to more than 5 jacks. The work should be leveled every 18 in. that the forms rise and any low parts should be jacked up to the level of the highest part. The vibration and nature of the platform make it almost impossible to level the forms with a dumpy-level, while the use of a carpenter's level is slow and laborious. The most practical method is to use a water-level consisting of a one-inch rubber hose having a glass tube projecting from each end. Before the jacking is started, each yoke should have a level-mark upon it; these marks can then be compared, by means of the water level, as the forms rise. In cold weather the water in the level should be saline to prevent freezing. Another method is to graduate steel flats and set them in the concrete so that the exact height of the work at any point can be determined with ease. The wall should also be plumbed or checked for vertical every 5 ft. because it is possible for the platform to be level while moving slightly sideways. This is generally the result of irregular jacking; the side raised first tends to move the platform towards the low side. This can also be detected by an examination of the distance between the bottom inside edge of the outside mold, and the concrete which it has just left. The scheme previously described for placing the wall in 8-in. lifts very greatly simplifies jacking and leveling, since a series of marks 8 in. apart may be made on the jack rods, and thus the crew can continue jacking until these marks are reached. The skilled laborer at this work in North America can attend to about 15 jacks when the forms are rising 4 ft. in 8 hr.

Reinforcement. The quantity of reinforcement required at various points in the walls must be determined theoretically, but the method of arranging these bars is a practical problem. When the concrete is to be placed in layers, horizontal bars should have a fixed vertical spacing The reinforcement then will be designed by the "constant space" method, dimensions of bars varying directly with the stresses. Except within the bottom 4 ft. of the walls, the reinforcement can never be set up in advance of concreting. The usual procedure when adopting the above constant space method, is to lay the steel on the concrete and to use as little tie-wire as possible. The reinforcement must be placed in position quickly and soon will be covered with concrete, so there is danger that some bars may be omitted. The best way to assist the supervisor in overcoming this difficulty is to simplify the arrangement of the reinforcing bars. The lack of space between the top of the molds and the lower cross member of the yokes causes

difficulty in threading long and bent bars into place. Thus, in spite of the waste of steel in laps, it is generally most economical to use bars no longer than 15 to 20 ft.* The splices in the horizontal rods

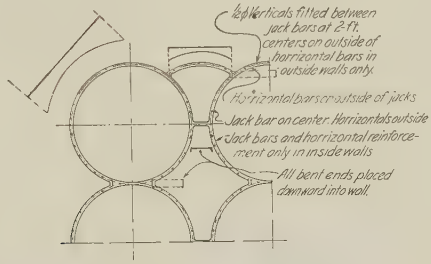


FIG. 18—ARRANGEMENT OF REINFORCEMENT FOR CIRCULAR BINS

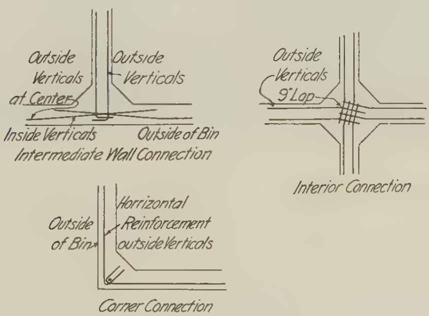


FIG. 19—DETAILS OF REINFORCEMENT FOR RECTANGULAR BINS

should not come directly over the splices in the course below but should be staggered very carefully. The speed of operation makes it advisable to have all main reinforcement greater than 3/16 in. in diam. and all short connecting pieces bent to the requisite shapes before elevating them to the platform. The method of reinforcing circular bin walls with a spiral, often used in stationary formwork, is not practical for sliding formwork. Because of the nature of the surface of sliding form concrete, there is danger in having the reinforcement too close to the exposed side of the wall as water may seep in through the hair-cracks, and, rusting the steel, cause it to expand and do serious damage. Thus it is not advisable to have the bars within 1 in.

*It is not necessary to limit the length of the reinforcing bars. Fewer laps will lead to less breaking out in the future and if the rods are properly fabricated before hoisting to the working platform, they can be put into place and wired with a minimum of work. Bars up to 30 ft. can be used very conveniently and should be wired to every vertical rod, and to the jack rods.—JOHN H. HEINDEL

of the exposed side of the wall, although there is no danger in having them within ½ in. from the inside face or from either face of the interior walls. Fig. 18 shows a simple method for reinforcing circular bin walls. The ends of the bars are not hooked or cogged but are given an extra length of lap because the hooks are troublesome when threading them through the yokes. Vertical bars are shown placed at 2-ft. centers between the jack bars in the outside walls. Fig. 19 shows details of reinforcement for the corner and interior connections in rectangular bins. The walls of circular bins may require 50 to 120

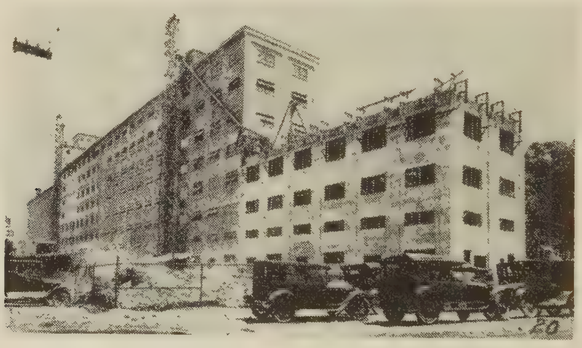


FIG. 20—CONSTRUCTION OF WASHBURN-CROSBY MILL IN KANSAS CITY
(Reproduced by permission of the Jones-Hettelsater Construction Co.)

lb. of reinforcing steel per cu. yd. of wall concrete. This variation is caused by differences in the bin diameters and heights which control the grain pressures. As a general rule for average sizes of bins, the quantity of reinforcement in North American practice is approximately 75 and 80 lb. per cu. yd., for circular and rectangular bin walls respectively.

Plant. The plant for sliding form work must differ from that used in stationary form work because the level of the working platform is continuously altering. For small work, the most economical scheme is to hoist the concrete with a jib projecting from the platform, and hence always at the correct level. When the hoist tower is adopted, provision must be made for raising the concrete hopper with ease. Because the jack rods and yokes project above the level of the platform, it is not practical to spout the concrete direct to the walls, and so the general procedure is to distribute the concrete with buggies or dump carts.

Miscellaneous Structures. The preceding comments on sliding form work are applicable not only to the construction of circular and rectangular bin walls, but also to the construction of the walls of buildings

in general. The walls, beams and columns of buildings can often be constructed quickly and economically as already described. The success of such work will depend upon the manner of making necessary modifications. The adaptation of sliding forms to this work is a recent development. Sliding forms will always show considerable economy on high but simple concrete structures for which the ratio of wall area to floor area is high. Fig. 20 shows the use of sliding forms in the construction of the Washburn-Crosby Mill in Kansas City. This work demonstrated that sliding forms could be used with economy on comparatively low buildings with dissimilar floor loads. Sliding forms are especially economical for the construction of head-houses or working houses which require bins because the forms for the internal bin walls can also be used for the beams and garner walls. The practical necessity for keeping external pilasters and wall panels continuous from top to bottom of the building may be accepted as an aid to beauty.

Walls, Pilasters, Columns and Beams. In general building work, the wall thickness is controlled by weather and working conditions rather than by bending moments. This minimum thickness for sliding form concrete should be 4½ to 5 in.* Where these walls are required to serve as bin walls for a part of their height, the dimensions of the bins should be adjusted so that a 4½ to 5-in. wall is sufficient to withstand the grain pressures. The formwork for the bin walls can then be used, without alterations or waste, for the wall panels above and below the bins. The frames forming the window openings must be made so that they fit inside the sliding forms and should be anchored to the reinforcement so that they will not be dragged out of position. The floor slabs are not placed until after the walls opposite that floor are finished but for purposes of stability and in order to accelerate the completion of the work, they should be placed as soon as possible after the sliding forms have passed. For the construction of this floor a key should be formed around the walls at floor level. This key may be formed by leaving a strip 4 in. deep by 1½ in. wide of dressed timber in the wall at the correct level and carefully anchoring it so the forms will slide past. To improve the connection between the wall and the floor, the faces of the key should be chipped with a rock-pick before the floor slab is placed. The beams, together with the pilasters and columns which support them, can be constructed very simply with sliding forms. The beams will be placed up to the underside of the slabs, and hence must be provided with numerous stirrups for making the connection to the slab, to obtain T-beam action. Side forms for

*This thickness is hardly enough for waterproofness of exterior walls and the interior walls generally are required to be thicker either for stiffness as arches or in buckling in vertical compression at the bottom.—Edmund Wilkes, Jr.

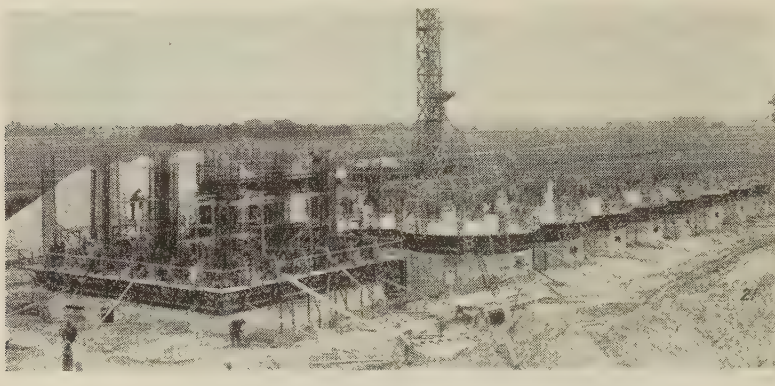


FIG. 21—SHELLABARGER ELEVATOR AND MILL CO'S. ELEVATOR AT SALINA, KANSAS
(Reproduced by permission of Macdonald Engineering Co.)

the beams should be constructed and carried upon the jack rods with the remainder of the formwork. The beam forms are not required between floors, and must then be stopped off from the pilasters and columns. The jack rods throughout these spaces will be in the open and must be braced every 4 to 5 ft. to prevent their bowing. From the standpoint of economy, it is essential to have the beams in the various floors vertically above each other, and all beams in any vertical projection should be the same width. As the forms arrive at the beam level, the stoppers are removed from the ends of the beam forms, and a support for the beam bottom is placed between the side forms. This beam bottom should be shored up from the top of the previous beam. The reinforcement may then be placed and the beam concreted for its full depth. This work causes a short delay in jacking, but as soon as the beams are filled, the jacking may be resumed. Fig. 21 shows the construction of the Shellabarger Elevator & Mill Co.'s elevator at Salina. The entire work was carried up simultaneously and interior beams and walls in the head-house were placed without delay to erection of main storage bins. The jack rods should pass through pipe sleeves at the beams and thus prevent the waste of steel and labor that otherwise would be incurred in cutting the projecting rods. The fillets which must be formed at the ends of beams determine the most economical shapes for pilasters or columns. Fig. 22(a) shows the detail of a pilaster including reinforcement and demonstrates the method of reducing the size, while Fig. 22(b) shows these details for an interior column. The reduction in size is made by inserting fillers in the forms, but the "beam faces" of columns and pilasters will always be the same width.

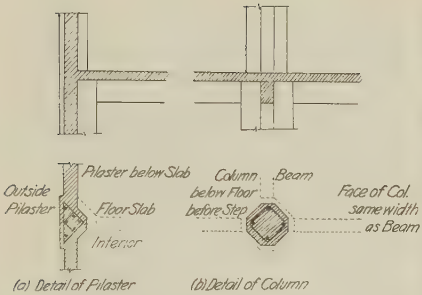
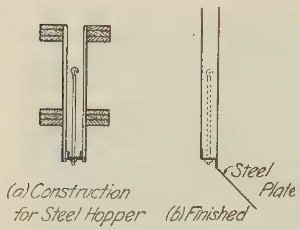


Fig. 22—(Above) Details of building connections

Fig. 23—Method for stopping-off an interior wall form

Overhead Bin Bottoms. Bottom slabs of working house bins are generally required at a considerable height above the ground because the prime object of these bins is to feed cleaning or separating machines generally placed below the bin bottoms. The sliding forms will then start at ground level but the interior bin wall forms will be stopped-off until the required level is reached. Fig. 23 shows a method for stopping off an interior wall form. The suspension of the bin bottom from these interior walls requires careful consideration on account of the large loads involved. When steel bin bottoms are to be used, the problem may be solved by leaving bolts projecting from the concrete, or else setting the top member of the hopper in place, as shown in Fig. 24. However, steel bottoms are generally more expensive than concrete so the schemes shown in Fig. 25 and 26 have been evolved. The bin bottom is designed as a horizontal slab of uniform thickness and the hoppering is made up with filling after the slab has been completed. The ends of hair-pin bars project down into a sand box, the bottom of which provides a level surface for shoring. These slabs are generally reinforced in both directions and the slab bars pass over the carrier bars, as shown in Fig. 25(b). A key should be left for these slabs around the inner face of the outside wall, as previously described, Although theoretically, this scheme may not appear ideal, it is commonly used in North America with satisfactory results. It is partially due to the success and economy of such methods for constructing



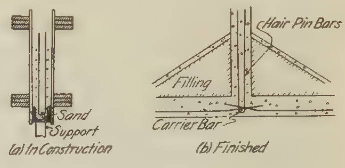


FIG. 24—(TOP, LEFT) SUSPENSION OF OVERHEAD STEEL BIN BOTTOMS

FIG. 25—(TOP, RIGHT) SUSPENSION OF OVERHEAD CONCRETE BIN BOTTOMS

FIG. 26— (LEFT) FORM FOR BOTTOM OF SUSPENDED INTERIOR WALL

Note that the effect of the sand box is obtained by letting the hairpin bars project down into the space between two boards which are kept apart with packing pieces.

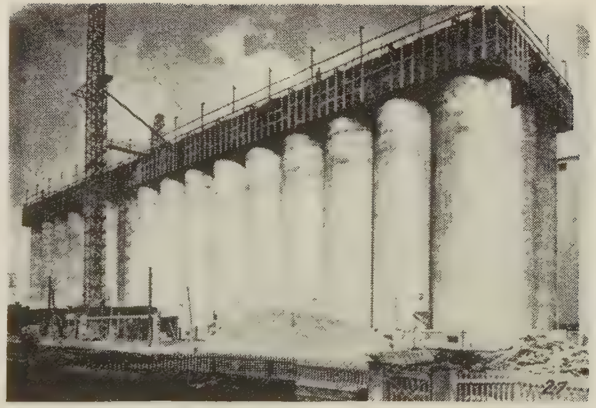
these bin bottoms, that the procedure of placing bins both above and below the cleaning machines is becoming increasingly popular. Thus the improved construction methods tend to eliminate the belt-elevators previously required for feeding and removing the grain from the cleaning machines.

Concrete Mixture. The success of sliding form work is largely dependent upon the aggregates used and the concrete mixture. The particles of the coarse aggregate generally should not be larger than 1 in. in diam. because there is a danger that they may become wedged between the forms and reinforcement, and thus destroy the face of the concrete. The particles of both sand and stone may be either smooth or sharp, provided they are proportioned with the cement to give a dense, well-graded mixture. Some builders contend that the addition of admixtures facilitates densifying the concrete, makes it waterproof and by smoothing it up enables the forms to slide more

easily. Apart from determining the strength of the concrete, the quantity of mixing water is of further vital importance. Excess water will seep away and destroy the surface of the wall, while if the concrete is too harsh or unworkable, the forms will not slide easily. The general practice is for the upper sections of the walls to be the same thickness as the lower sections, and hence, since they carry less load, a saving may be effected in the quantity of cement by adjusting the proportions of the concrete mixture to the stresses. However, there are practical limitations to the proportions of materials which will make the consistency of the concrete suitable for sliding form work. These proportions give a concrete with an ultimate strength of 2,000 p.s.i. This is the poorest concrete which should be used for sliding form work but the lower part of the walls may be quite successfully designed for the smoother, richer concrete, which stresses up to 3,000 p.s.i. Thus a saving may be effected by using higher stresses with thinner walls of stronger concrete. Some engineers contend that no mixtures testing less than 2500 p.s.i. in compression at 28 days, should be used in parts exposed to the weather. There is no doubt that the walls frequently would be improved by making them thinner and of better concrete. Alteration of the concrete proportions during construction interferes with the free course of the work increasing the time and labor required. However, the advantage to be gained by avoiding excessively thick sections caused by the high stresses in the lower portions of the structure, should be carefully considered.

The danger of decreasing the strength of the concrete through exposing it to the weather a few hours after placing must also receive careful attention. This danger is generally due to the concrete hardening too quickly, so that in warm weather some of the more careful construction companies drag damp tarpaulins up over the new concrete. These tarpaulins, however, are likely to flap in a high wind, and tend to ruin the texture and weathering qualities of an exterior wall. Sprinkling is apt to wash grooves in the finish. Some advantage recently has been gained by the use of a light covered frame as a sun-shade below the form. There is need for improvement in curing sliding form walls and it will doubtless come with due attention.

Weather Conditions. Jacking and concreting of sliding form work are sometimes stopped by heavy rains or storms. The forms should then be raised clear of the concrete and left until the weather improves. In North America, however, the builder is very often required to commence and carry out the work during severe winter weather. This is necessarily expensive but serves to minimize the slack season for



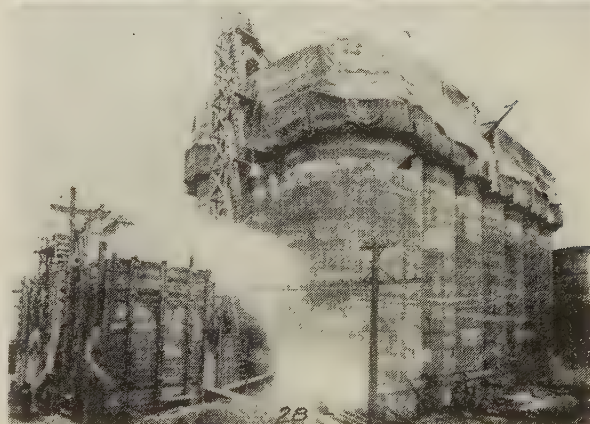


FIG. 27—(TOP, LEFT) CONSTRUCTION OF RED STAR MILLING CO. ELEVATOR AT WICHITA, KANSAS, DURING WINTER
(Reproduced by permission of Jones-Hettelsater Construction Co.)

FIG. 28—(BELOW, LEFT) CONSTRUCTION OF BINS FOR THE HAWKEYE CEMENT CO. AT DES MOINES, IOWA
(Reproduced by permission of the Macdonald Engineering Co.)

FIG. 29—(ABOVE) VERTICAL JOINTS IN WALLS
Note method of providing for a key for the wall which starts part way up the main walls

construction work and insures that the elevator will be ready for the first harvest. The problems of heating concrete materials, preventing concrete from freezing, and protecting workers from the elements, are similar for all construction work carried out in cold weather. Applying the heat so that the forms will slide satisfactorily is, however, a special development. The heat is usually supplied from coke burning salamanders, and the inside of bin walls can be heated very simply by placing one of these heaters under each bin outlet. The warm gas will then rise and the top deck will prevent its escape. This scheme, unfortunately, prevents any examination of the inside molds or face of the concrete because the fumes contain poisonous carbon monoxide. The outside hanging scaffold should be enclosed and this may be done by means of sheeting, as shown in Fig. 27. or by the use of tarpaulins as shown in Fig. 28.

Provision for Connections. Reference has already been made to the methods of forming keys and providing for window openings, in sliding form walls. Another method is to place reinforcing bars so that one end is cogged into the wall, and the remainder of the bar lies along the face of the form. The bars then can be bent out from the wall after the forms have passed. Provision for continuous vertical joints can be made by sliding the forms past a strip of timber through which the requisite bars project as in Fig. 29. As it is practically impossible to place the bars, holes, or keys in their exact positions, these connections are generally expensive and unsatisfactory. The designer should not plan any such connections which can be avoided.

*Stripping and Reusing the Forms**. When the forms have reached the top of the walls and the jacking is finished, the weight of the forms is generally transferred from the jack rods to bolts or bearers inserted through holes left in the walls below the walings. The location of these holes must be carefully planned from the proposed method of supporting the forms. When the weight of the forms is thus taken off the jacks, the short upper lengths of jack rods may be removed and, if required, the yokes also. When the yokes are not required elsewhere they are sometimes left in position until after the slab over the bins is concreted. Small boxes then are set around the vertical members of the yokes so that they can easily be withdrawn after the slab is poured. The boxes then are removed and the holes filled with concrete. The yokes and various other sections of the formwork are frequently required elsewhere as soon as possible and hence are removed before the bin top slab is concreted. Stripping and removing the bin forms

*"Stripping and Reusing the Forms" has been added to the original report by the author at the suggestion of J. H. Heindel.—Author

must be properly organized else they may prove to be very expensive items. The removal of the outside forms presents no great difficulties or special features but it is usual to leave holes in certain planned positions through the bin top slab in order to facilitate the suspension of a stripping scaffold inside the bins. There are types of construction for the formwork of the slab over the bins which enable the forms to be removed intact but generally it is only practicable to salvage the yokes, larger members and sections of the walings. The cost of labor in stripping and the value of the used timber should be carefully investigated before it is decided to drop and waste any sections.

Ensilage Bins. Many difficulties will beset the small country builder who endeavors to construct reinforced concrete ensilage bins using stationary forms. These dangers will be considerably increased if he tries to use sliding forms, because he then deals with a machine which must be set up accurately, and operated carefully.

Quality of Sliding Form Concrete. After the concrete is placed it is spaded, then the forms rise and tend to drag it up with them. The concrete then receives further spading, the forms again rise, and these disturbances occur several times while the concrete is endeavoring to harden. Finally, when the forms expose the wall, the concrete is rubbed down with a wooden float. Very often the sun is shining on the concrete within 8 hr. after placing, due to carelessness in not protecting the surface. The shocks to the surface of the concrete and the early removal of the restraining forms, do considerable damage and result in a surface, covered with hair-cracks, which cannot generally withstand the weather.*

Designing for Sliding Forms. The jack rods in sliding form walls are usually too far apart to act as vertical reinforcement. It is usual to place at least ½-in. diam. vertical bars at 2 ft. centers between the jack rods in all external walls to prevent the formation of exposed horizontal cracks. These verticals should also be placed throughout the interior walls, but usually are omitted because they interfere with the work on the platform. The author's previous comments on the

*The examination of a large number of such structures by the writer has failed to disclose evidence of disintegration due to weathering or to defects resulting from causes mentioned by the author. It is true that there are scattered over the country too many examples of cracked bin walls, but a check of soil bearing conditions at each structure generally points to racking action caused by unequal settlement of the foundation. As evidence of the high quality of well constructed sliding form walls, the writer has in mind a molasses tank erected in Kansas City, Missouri. This tank is 30 ft. in diameter, and has 12-in. walls subject to a head of 42 ft. of a fluid weighing 90 lb. per cu. ft. During construction this tank was floated both outside and inside. There is no evidence of weathering or disintegration due to the checking of the surface or to any injury to the body of the concrete during construction. The walls, in spite of severe temperature variations, and the fillings and emptyings of the tanks, perform their elastic function excellently. Another example is a group of grain tanks in Sherman, Texas, built about 15 years ago. The walls show no checking or weathering whatever. Numerous such instances can be cited to the same effect. Research and practice have demonstrated in recent years that vibration and a certain amount of reworking for a reasonable period after mixing are beneficial rather than injurious. Therefore, this writer feels that sliding form concrete is worthy of fully as high a rating as that produced with stationary forms.—EDMUND WILKES, JR.

quality of sliding form concrete suffice to explain the reason that sliding form walls generally must be designed thicker than those to be constructed with stationary forms. Sliding form walls may be designed ½ in. thicker than stationary form walls on the reasoning that the concrete to a depth of ¼ in. from the surface is incapable of taking any load on account of hair-cracks.

It must be remembered when testing samples of sliding form wall concrete, that these samples should be cured under conditions similar to those experienced by the wall. The reinforcement and field work should be made as simple as possible by the designer. In sliding form work there is a great tendency for the workers to omit, for instance, an occasional intricate reinforcing bar. The constant movement, makes it difficult to note such omissions until too late. Not only are these "adjustments" made by the careless laborer, but they are too often made by over-confident supervisors. This state of affairs is regrettable, but nevertheless must not be ignored.

These severe statements regarding sliding form concrete are furnished for the benefit of designers who provide plans for builders inexperienced in sliding form construction rather than for specialists who design and build knowing the exact limitations of their outdoor staff. It is hoped that the criticism will assist the improvement of sliding form work rather than create prejudice against an essentially good form of construction in which the author has every confidence.

Possible Improvements. The waste of concrete is economically a serious disadvantage of the existing sliding form system. Surprisingly little effort has been made in North America to eliminate this waste although there are many ingenious schemes for saving labor in less important directions. The pump jack and the methods for raising the forms by power from a line shaft or by hydraulic pressure, have been developed to minimize the labor of jacking. The cost of labor involved in mixing and placing the concrete, however, is 4 times that of jacking, and yet very few efforts have been made to reduce the volume of concrete. The scheme which has been described for saving cement in the upper sections of the wall, fails to reduce the waste of sand, stone and labor.* Although difficult to predict the tendency of construction methods, it is very probable that serious efforts will be made to overcome this waste in the immediate future. The North American is at present too satisfied with his own methods and prepared to sacrifice materials in order to save labor. Thus the development

*This does very well for most cases within the range of ordinary practice in North America.—Edmund Wilkes, Jr.

may be expected to occur in the Argentine, where concrete materials are expensive, and labor costs comparatively low.

Variation of Wall Thicknesses. The requisite wall thickness at the bottom of a circular bin is theoretically determined from the compressive stresses. The thickness at the top where the loads are small, is controlled by the minimum necessary for constructional purposes or for keeping the bin weather-proof. The ideal bin wall should taper uniformly from the calculated thickness at the bottom, to the point at which the thickness, as determined from the compressive stresses, coincides with that required by the weather conditions. The wall should then continue to the top with uniform thickness. Fig. 30 shows diagramatically the methods by which the thickness of the wall of a 20 ft. diam. bin, 90 ft. high can be varied to obtain economy, and in

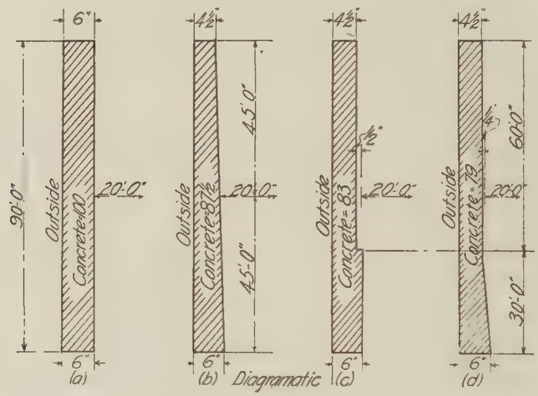


FIG. 30—VARIATION OF WALL THICKNESSES FOR BINS WITH THE SAME CAPACITY

each case, the outside of the bin wall is kept vertical for the sake of appearance.

The case of a single bin is taken but where there are groups of bins, the designer must satisfy himself that the thin walls at the top can withstand the arch action from the interstice or interspace bins. It must be remembered that the grain pressure in these smaller bins does not decrease very considerably until very near the top. Fig. 30(a) shows the usual sliding form wall of constant thickness, which for purposes of comparison is considered as 6 in. Fig. 30(b) shows a constant taper from 6 in. at the bottom to 4½ in. at the top. This method involves extra work which will, however, be uniform for the constant taper and the saving in concrete will be approximately 12½ per cent when compared with the walls of the previous bin. Fig. 30(c)

shows the method of stepping the wall. The bottom thickness must usually be carried up for one third the height and the saving in concrete by comparison with the wall shown in Fig. 30(a) then will be approximately 17 per cent. This stepping of the wall necessitates additional labor only at the point where the thickness is changed. Fig. 30(d) shows the theoretically ideal scheme and effects a saving in concrete of 21 per cent on the first method but necessitates extra labor for the first third of the height. An additional saving is effected by decreasing the volume of concrete in the bin walls, because this reduces the load on columns under the bins and also the loads on the foundation.

A smaller percentage of saving can be effected by reducing the thicknesses of bin walls that are designed for bending moments. The lengths of the sides of these bins are usually, for economical purposes, less than 15 ft. and hence there is little variation of grain pressure in the deep bins. The principal saving can generally be effected only over the top third of the height, whereas with bin walls that are designed for compressive stresses, the principal saving extends over two-thirds of the height.

Sliding Forms for Reducing Thicknesses of Walls. The most obvious method for reducing the thickness of sliding form walls is shown in Fig. 30(c). When concreting is completed to the level of the step, the forms should be jacked up until the bottom of the molds is level with the top of this concrete. The forms can then be adjusted by re-building the inside shutters, or by fitting a filler piece inside the molds. The construction of the upper section of the wall may then proceed, but considerable care must be exercised in the balance of the yokes since the jack bars cannot be central in both the lower and upper parts of the wall. The stopping of jacking while the alteration is made, can be turned to advantage by placing the hopper filling during this period. There is no doubt that the cost of extra labor in making such alterations will be more than repaid by the saving of 17 per cent in the bin wall concrete. This scheme, however, will result in unnecessary horizontal lines and cracks with the usual bad appearance and consequent weathering resulting from this procedure.

The form with an adjustable yoke may be used to construct walls as shown in Fig. 30(b) and 30(d). The method shown in the former involves uniform work at closing in the forms from start to finish, but the additional saving of 8½ per cent in the quantity of concrete will make the scheme shown in the latter figure the more economical. The vertical arms of each yoke may be connected to act as "parallels" so when the inside shutter is lowered, it will approach the outside one.

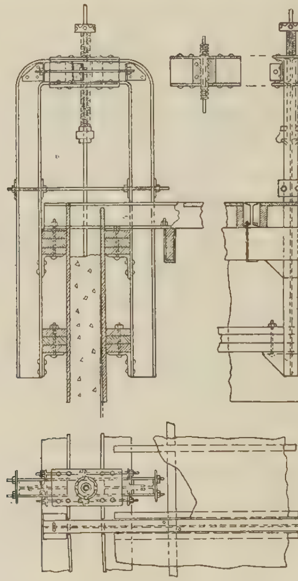


Fig. 31—Argentine patent drawing no. 32,438

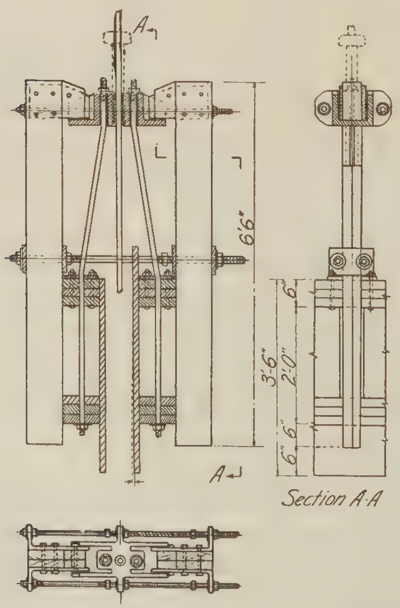


Fig. 32—Timber yoke for adjustable forms

Another method for drawing the shutters together is by directly applying pressure through a screw. These adjustable yokes should be fitted with bolts for drawing the jack rods into the central position. The working platform must be attached to the outside molds and hold them vertical, thus leaving the inside shutters free to control the thickness of the wall.

Adjustable Sliding Forms. Fig. 31 shows the self-explanatory drawing for Argentine Patent No. 32,438 granted the writer in 1929. The cross-head and nut are constructed to take the entire weight of the forms without being rigidly connected to the side members of the yoke. The relative positions of these members can be controlled by means of the two sets of bolts, while the top bolts also restrain the movement of the cross-head and hence the jack-rod. The platform is attached to the outside mold, and the steel beams for supporting this platform follow the procedure already discussed. These steel beams are even more economical with the adjustable forms because the platform is supported on the outside molds and thus the span of the beams is increased. These beams must be packed up off the outside walings to which they are attached, to assist in the formation of the cornice, and in order to leave the inside mold free. The inside mold requires a greater slope than is necessary with the usual sliding forms, to enable it to be pulled with ease towards the outside. The shutters should be drawn together simultaneously with the jacking, and the supervising engineer must know the ratio between the jack screw and the tapering screw in order to obtain the desired result. The yokes should be constructed of steel for reasons of economy, but Fig. 32 shows a method for making them of timber. Fig. 33 shows the design for the platform of a 20 ft. diam. bin, and Fig. 34 shows the platform for a 23 ft. diam. bin. An account of the necessity for increasing the inside diameter of the bins, all the inside walings should be sawn through at points midway between the jack-rods after the entire framework is balanced. Then as the forms rise, wedge strips of timber are inserted as required. The interstice bin can be covered with fixed flooring and the sections of wall which are common to 2 bins, must be tapered or stepped on both sides. Thus in order that these sections of wall may finish at the top with the same thickness as the remainder, they must start with the extra thickness at the bottom. The horizontal reinforcement bars should not be excessively long, as generally it is preferable to waste steel in laps than to waste time in placing the steel. A ring of steel in a 20 ft. diameter bin very conveniently may consist of 4 bars. The inconvenience of threading these bars under the ends of the steel joists is

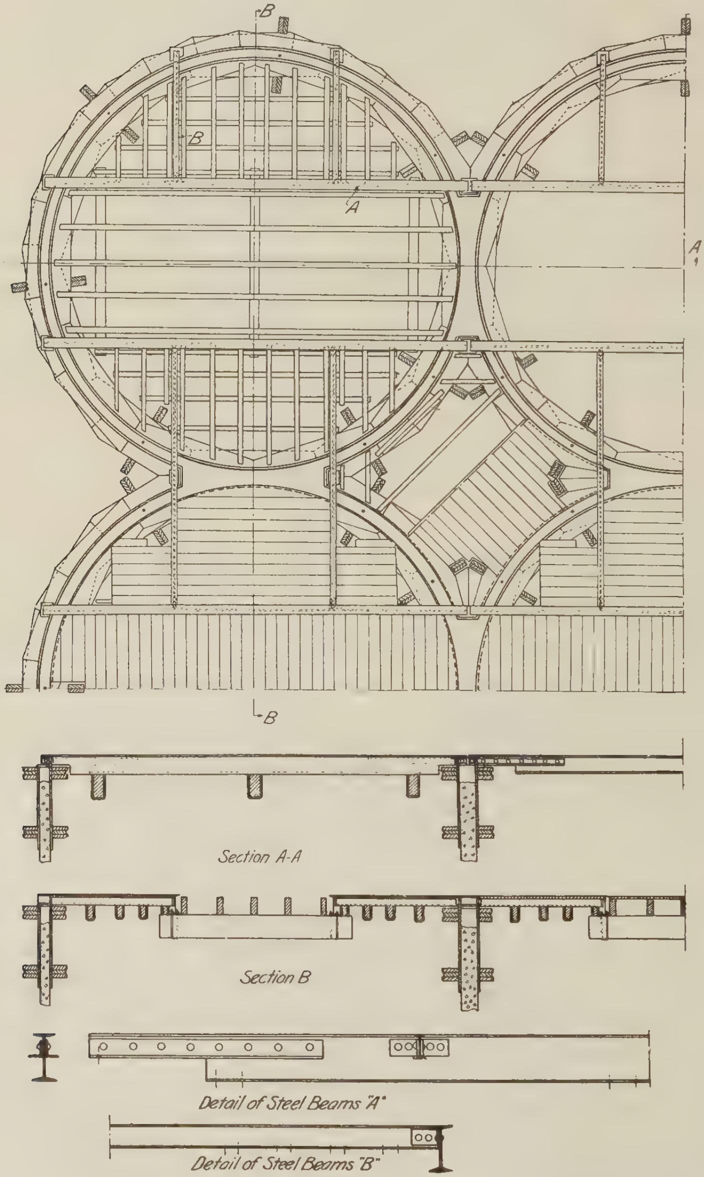
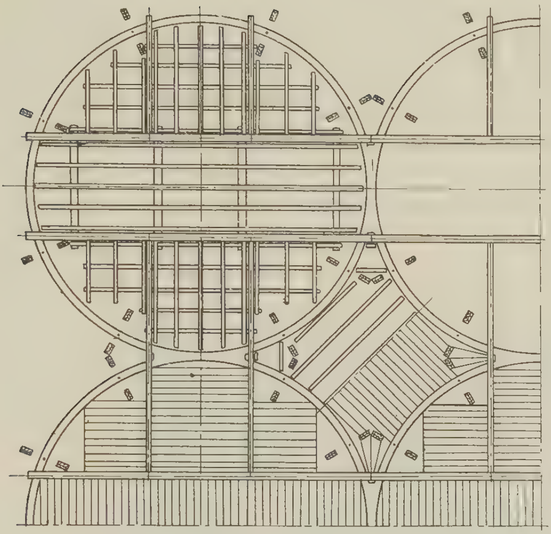


FIG. 33—PLATFORM FOR 20-FT. DIAMETER BIN USING ADJUSTABLE FORMS



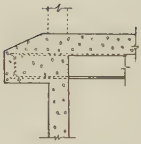


FIG. 34—PLATFORM FOR 23-FT. DIAMETER BIN USING ADJUSTABLE FORMS

FIG. 35—(ABOVE) DETAILS OF CORNICE

counteracted by the fact that the oustide shutter is lower than the inside one. On account of this low outside shutter, a special tray will be necessary for filling the wall with concrete. Fig 35 shows a detail of the cornice, and indicates the method for encasing the projecting ends of the steel beams in concrete. The wall should be concreted up in short lengths under these beams, so that the load on the beams is transmitted to the wall when the packing pieces mentioned above are removed from under the ends. The above comments have been made with reference to the walls of circular bins but they are also applicable to straight walls.*

ADAPTATION OF METHODS TO SOUTH AMERICAN CONDITIONS

Construction of the Buenos Aires Great Southern Railway elevator at Ingeniero White, the port for Bahia Blanca, has furnished interesting data relative to sliding form work in South America. The main storage bins consist of 12 rows of 9 circular bins, 18 ft. inside diam. and 90 ft. high. Working house bins consist of 12 rows of 6 circular bins, 14 ft. 9 in. inside diam. and 60 ft. high. As considerable difficulty was expected in obtaining sufficient foremen and in organizing inexperienced peons for construction of the main storage and working house bins in complete units, it was decided to build the

*A sliding adjustable form which tapers the wall would seem to be the most economical as it should require less labor and less delay in the construction and avoid horizontal cracks. However, the economy and efficiency of adjustable sliding forms cannot be determined until they have been used on several jobs of different types as there may be "bugs" in this procedure which could only be worked out in actual use but there is certainly no doubt that an attempt should be made to work it out particularly for use where materials are expensive and labor cheap.—J. H. HEINDEL

main storage bins in 3 sections and then the working house bins in 2 sections. This procedure also had the advantage of requiring considerably less jacks and very much simpler and smaller concreting plant. Each section of the main storage bins had 225 jacks and of the working house bins, 305 jacks. Fig 36 shows an aer'al view of the building after the 3 sections of the main storage bins had been completed and the first section of the working house bins was nearing completion. Fig. 37 shows the relative rates of construction for the

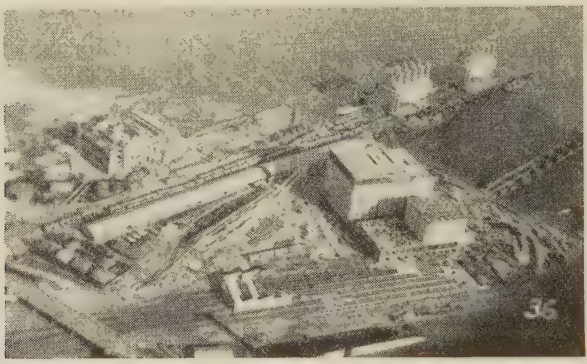


FIG. 36—BUENOS AIRES GREAT SOUTHERN RAILWAY ELEVATOR, INGENIERO WHITE, ARGENTINA
(Reproduced by permission of Messrs. Henry Simon Ltd.)

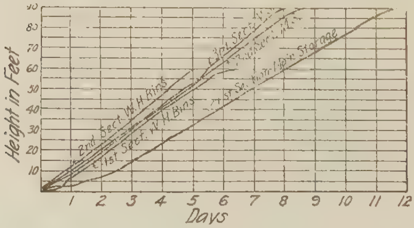


FIG. 37—SPEED OF SLIDING FORMWORK
Main storage bins constructed in three sections during winter and then the working house bins constructed in two sections during the summer

sections of bins and demonstrates that invaluable experience in organization and operation were obtained on the first section of the main storage bins. The second section of the main storage bins was constructed with a saving of 25 per cent on the total working hours, 26

per cent on the hours of concreting and 29 per cent on the hours of jacking. The third section of the main storage bins was constructed with a further saving of 8 per cent on the total working hours, 9 per cent on the hours of concreting and 8 per cent on the hours of jacking on the second section. The working house bins cannot be compared in such a simple manner because there were delays for the placing of numerous boxes for openings and for commencement of walls which started part way up the main walls. The increased speed for working house bins as compared with the much simpler main storage bins is largely due to the fact that this work was carried out in summer whereas the main storage walls were erected in winter, and partly due to previous experience.

CONCLUSION

The builder should gain local experience both in stationary and sliding forms and not blindly accept either system. When the controlling factor is cost, and not the time for completing the walls, the quantity of timber in both stationary and sliding forms will be approximately the same. The main economical advantage of the sliding forms is the low cost for raising them, whereas the economical advantage of the stationary forms is the saving of concrete. This saving is not only in the tapering of the walls, but in the adoption of generally thinner walls because of the superior nature of the concrete. However, when builders learn economical methods for reducing the thickness of sliding form walls, there is no doubt that these forms will supersede the stationary forms for all work on bin walls. At present, when materials are very expensive, and labor rates comparatively low, as in the Argentine, the stationary forms often prove to be the more economical. In North America, however, the sliding forms are the more economical because speed is important and the material costs are low as compared with the labor rates. Thus, practically all the North American designs for buildings such as grain elevators, are prepared for sliding form work, and the builder who is not free to make his own plans, must, for economy, use the sliding form construction system. A suitable design for sliding form work seldom will be economical for stationary forms. In a similar manner, bin designs prepared by Continental engineers are generally designed for stationary formwork and these plans will be impractical for sliding forms. Designers of this class of work should be thoroughly conversant with the most economical construction methods and capable of gaining the maximum combined economy in

both concrete and formwork, while the builder should be capable of planning in advance every detail of the operation. The designer and the builder should cooperate.

A supplemental report on "Sliding Form Work," by W. R. Sproul, member of the committee will be published in this JOURNAL *for February. Subsequently readers are referred to the* JOURNAL *for June, 1933, for discussion which may develop. Such discussion should reach the Secretary by April 1, 1933.*

Discussion of a Paper by Mark Morris:

"The Mortar Voids Method of Designing Concrete Mixtures"*

Editor's Note—On behalf of Mr. Morris, author of the original paper, attention is directed to errors in printing as follows:

p. 12, next to last line of last complete paragraph, "$a/c + 1.0$, and $a/c = 5.0$" should be "$a/c = 1.0$, and $a/c = 5.0$."

p. 13, line 12, "represent" should be "represents."

p. 21, line 25, "design" should be "designs."

F. H. Jackson†—Mr. Morris has presented a very interesting discussion of the results being secured in Iowa through the application of the Talbot-Richart mortar-voids theory of designing concrete mixtures. He has cited impressive figures to indicate economic advantages through the use of mixtures designed to produce concrete of a definite required quality in place of the old style arbitrary method of proportioning.

I shall not attempt discussion of the technique of the mortar-voids determination, but confine my remarks to one or two points regarding the utilization of the data to the general problem of designing concrete pavement mixtures.

In the first place, I question very much the use of crushing strength as the basis of design when the concrete is to be used in pavement construction. It is true that adequate resistance to compression is essential. However, everyday experience teaches us that a concrete pavement is far more apt to fail in tension or flexure than in compression, due to the fact that the tensile or flexural strength of the concrete is much more likely to be exceeded by reason of contraction of the slab or the application of excessive wheel loads, than is the crushing strength. As a matter of fact, failures in compression are confined almost entirely to occasional "blow-ups" or local crushing at

*Presented at the 28th Annual Convention, March 1-4, 1932, Washington, D. C. Published, Journal Amer. Concrete Inst., Sept., 1932. *Proceedings*, V. 28, p. 9.

†U. S. Bureau of Public Roads.

joints due to expansion. By and large, this type of failure is comparatively rare. It would seem therefore that designing paving mixtures on the basis of crushing strength is justified only on one condition; namely, that, for the types of materials available, the flexural and tensile strength of the concrete varies in proportion to its crushing strength; in other words, that designing for a definite crushing strength will automatically insure a correspondingly definite resistance to flexure. I believe that, within reasonable limits, this condition applies in the State of Iowa. So far as conditions in Iowa are concerned, therefore, I can have no quarrel with the method of design proposed. However, experience has indicated that for many types of aggregates a fixed ratio between crushing and flexural strength does not hold. I should like to illustrate this by reference to the problem of designing a concrete paving mixture for the Mount Vernon Memorial Highway.

It so happened that, while practically only one type of sand was available for use on this job, there were three distinctly different types of coarse aggregate available. One of these produced a concrete very low in flexure as compared to its crushing strength while the other two produced concrete having high flexural strength compared to crushing strength.

Repeated tests have shown that, for the same water-cement ratio, concrete containing the high strength aggregate will have a flexural strength approximately 30 per cent higher than concrete containing the low strength aggregate and that, to secure concrete of equivalent flexural strength, a differential of nearly 0.2 in water-cement ratio must be made in favor of the higher strength materials. Bear in mind that the same fine aggregate is used in both cases, and that therefore the same mortar voids relations would apply in each case. It is obvious, therefore, that a design based on mortar-voids studies alone, while it might have produced concrete of equal crushing strength, would have resulted in widely different flexural and tensile strengths. Bear in mind, also, that both the so-called "Older" corner formula for computing pavement thickness as well as the Westergaard theory assumes a definite flexural strength for the concrete to be used.

In the case cited, it was found that the difference in the strength characteristics of the coarse aggregates warranted a differential of one sack of cement per cubic yard of concrete. On this basis the specifications were set so as to require 7 sacks per cubic yard for the lower strength material and 6 sacks for the higher strength materials. The contractor chose the 7-sack proportion and it is interesting to note that, whereas the structural design called for a modulus of rupture of

600 p.s.i. at 28 days, the average of all tests of concrete beams cast on the job was 615 p.s.i. The average crushing strength, on the other hand, was 4,895 p.s.i. It is quite obvious that if the mortar voids method of proportioning had been used in this case and the concrete designed for, say, 4,000 pounds crushing strength, the actual modulus of rupture would have fallen far below the 600 pounds required by the design of the pavement. This case is cited simply to show that, where aggregates producing concrete having low flexural strength are encountered, some provision must be made for compensating for this low strength. I know of no better way of doing this than to lower the water-cement ratio until the desired strength is secured.

I should like to say also that the aggregate problem just illustrated is by no means a local one. Aggregates of the type indicated are widely distributed. Differences of this nature are probably less marked in some of the Mississippi Valley States than elsewhere. Where they do occur, I believe that we should recognize them and provide a method of design which will take such factors into consideration.

It seems to me that the simplest, most direct and most practical method of determining the proportions required for any given strength is by the water-cement ratio trial method. This was the method of design used in the Mount Vernon Highway work. It is also being used by certain of the state highway departments for designing pavement concrete. The method, expressed in the simplest terms, simply provides for a determination of the particular water-cement ratio required to produce concrete of the specified strength for each combination of aggregates, both fine and coarse, available for use. The method may, of course, be applied to studies of the crushing strength as well as flexural strength and may also be extended if desired to investigate the influences of particular cements upon strength.

There can be very little question as to the correctness of the fundamental relations between mortar strengths and voids as revealed by the mortar voids tests. These seem to be well established. However, Mr. Morris questions the use of voids-strength relations based on average results for a group of sands and prefers to use curves derived from studies of each particular sand. This necessitates strength tests on each sand and if this is necessary I am wondering why we can not go just one step farther and make strength tests on the concrete instead of assuming that the mortar voids strength relations tell the entire story and that the coarse aggregate has nothing to do with it.

So long as we persist in building concrete roads as we do at present, that is, simply spreading the concrete on the subgrade and smoothing

it off without any compaction to speak of, we must avoid the use of very dry mixtures in order to prevent honeycomb. For this reason, it is necessary to use plastic mixtures having fairly high relative water contents. For this type of mix the voids-cement ratio corresponds very closely to the water-cement ratio, so that relations established as the result of studies of one ratio would apply with equal force to the other.

From the standpoint of simplicity and adaptability to everyday construction operations I believe that the use of the water-cement ratio is preferable to voids-cement ratio. For the type of mixtures in which the highway engineer is interested, tests to determine the relation between strength and water-cement ratio performed on concrete specimens are more satisfactory we believe than the somewhat complicated series of laboratory tests on mortar specimens which it is necessary to make in order to establish mortar-voids strength relations for any given sand.

*Bailey Tremper** (by letter): Attention is called to Mr Morris' statement on page 12, "with normal coarse aggreates the strength of the mortar represents the strength of the concrete."

The Department of Highways of the State of Washington has for years been working on the problem of determining the concrete making properties of coarse aggregates. Our experience has led to the conclusion that none of the commonly specified tests for coarse aggregate will determine the strength of the concrete accurately. As a result of this experience we have devised a test which compares the concrete strength of the coarse aggregate in question with that of the gravel from Steilacoom, Wn. The two coarse aggregates are screened and recombined to the same grading, carefully graded sand from Steilacoom is used in both cases, and the concrete is made up with the same cement, the same slump, and the same proportions by absolute volume. As a general rule the cement contents have been between 1.66 and 1.68 bbls. per cu. yd. Concrete specimens containing the Steilacoom gravel and the coarse aggregate in question are made on at least four different days in order to secure reliable results. About forty coarse aggregates have been so tested. The majority have given strength ratios between 95 and 100 per cent. There have, however, been numerous exceptions and the following tabulation gives the results of some tests in which the coarse aggregate has given results varying considerably from the average:

*Engineer of Tests, Department of Highways, Olympia, Washington.

Coarse Aggregate	Water Used G.P.S.*	Relative Strength Compared to Steilacoom Gravel—Per Cent	
		Compressive Strength 6 x 12 in. Cyls.	Flexural Strength Beams
Gravel A	4.57	104	103
Gravel B	4.57	89	82
Gravel C	4.44	108	103
Gravel D	4.47	94	93
Gravel E	4.30	85	76
Crushed Rock A	5.25	75	—
Gravel F	4.50	87	—
Gravel G	4.65	90	70
Crushed Rock B	4.60	87	—

*Gallons of water per sack of cement added to room-dry aggregates.

It is evident that the mortar was the same in all cases yet the resulting concretes varied greatly in strength. These tests indicate that the failure to take any account of the characteristics of the coarse aggregate is a serious handicap to the use of the mortar-voids method. It may be argued that those coarse aggregates giving strengths widely different from the general average should not be classed as "normal." However, with a few exceptions, there was nothing disclosed by the usual tests for cleanness, coating, soft fragments, or hardness to warrant their classification as abnormal. The concretes contained sufficient mortar to provide a substantial excess over the voids in the coarse aggregate and they all possessed good workability so it is not believed the percentage of air voids was excessive in any case.

With differences in strength of as much as 30 per cent resulting from the use of different coarse aggregates it does not seem possible to predetermine concrete strengths from a consideration of mortar characteristics alone.

Readers are referred to the JOURNAL *for May 1933 (this volume) for concluding discussion.*

Discussion of Progress Reports of Committee 105:

"REINFORCED CONCRETE COLUMN INVESTIGATION"

EDITOR'S NOTE: Continuation of the discussion of progress reports on the Reinforced Concrete Column Investigation from the JOURNAL for Sept. 1932 (this volume, p. 53), is further postponed. A further, possibly a final, report of Committee 105 is scheduled for presentation at the 29th Annual Convention, Feb. 21-24, 1933, and concluding discussion will be scheduled when the new report is published.

Discussion of Report of Committee 407:

"Painting on Concrete Surfaces"

EDITOR'S NOTE: The report of Committee 407, presented by the Author-Chairman, F. O. Anderegg in the JOURNAL for Sept. 1932 (this volume), was to have been discussed in this issue. Discussion is scheduled by the Program Committee for the 29th Annual Convention next month and it will be published in the April, 1933, JOURNAL.

Concreting Problems—Chats Falls Power Development*

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INTRODUCTION

The Chats Falls hydro-electric development is on the Ottawa river, where the river falls from Chats Lake into Lake Deschenes, about 35 miles above the City of Ottawa. At this point there is a natural head of about 35 feet which, with the water available, permits the development of some 224,000 horsepower.

The development consists essentially of a concrete dam and sluices some 15,000 ft. long and averaging 20 ft. high, and a powerhouse, 500 ft. long, 72 ft. wide, and 133 ft. high, and required altogether some 254,000 cu. yd. of concrete in its construction. Concreting, begun June 19, 1930, was completed Nov. 13, 1931.

Three outstanding features of the work are of particular interest to those dealing with concrete:

(1) The completeness of the system of control beginning with the aggregates and following through the proportioning, mixing, transportation and placing, ending only when the concrete was cured.

(2) The great care used in placing concrete, particularly the large blocks in one continuous operation to heights as great as 60 ft. without horizontal construction joints.

(3) The methods of winter concreting, including the use of recording thermometers during winter weather to determine the amount and type of protection required to maintain proper curing conditions for the concrete.

The system of control, embodying no unusual features, was notable principally for the completeness with which it covered essentials and followed the principle that the maintenance of quality is not to be obtained by the adoption of some fancy test or mathematical design of mixtures, but is rather a co-ordinated effort to provide a smooth

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flow of uniform materials through the concrete mixer and their handling in such a way as not to impair the quality of the concrete. This, of course, implies a system of checks to see that the desired performance is being obtained, which in turn necessitates inspection, but co-ordination and co-operation, and not the police spirit, was the motive actuating the inspection of this job. Construction and inspection staffs worked together on this basis.

The inspection staff consisted of a chief inspector, resident inspectors at the cement mill and sand pits, six inspectors looking after the mixing and placing, and a laboratory assistant. The duties of these men will be indicated in the brief description of the salient points at which control of the concrete and its ingredients was exercised.

FINE AGGREGATE

Having the requirements for the maintenance of quality in mind, the engineers added to the usual exploratory work required to obtain suitable fine aggregates in an undeveloped country, a very thorough test pitting of the property from which the sand was to come. Twenty-five test pits sunk 100 ft. apart, to a depth of 30 ft., not only proved the sand satisfactory in quality, but gave accurate information on its quantity and gradation at any place and guided pit operation to produce the grading required.

The sand varying in the area excavated from a fineness modulus of 1.79 to 3.13, a resident inspector was stationed at the pit, who continuously sampled and tested the deposit and shipments, and directed the loading. He was instructed to obtain as nearly as possible, sand having a fineness modulus of 2.30, which often made it necessary that each car be loaded from several different points in the pit. Typical analyses of the sand are shown in Table 1.

TABLE 1—SIEVE ANALYSIS OF FINE AGGREGATE

Size of Sieve	Per Cent Passing:	
	Average of Pit	Average of Job
4	100	100
8	93	94
14	78	84
28	53	61
48	23	25
100	4.9	4
Fineness Modulus	2.49	2.32
Weight per cu. ft.	110	110.1
Specific Gravity	2.65	2.65
Per Cent Absorption	0.3	0.3

COARSE AGGREGATE

Coarse aggregate was made on the job from crushed rock from the powerhouse excavation. Here, too, every effort was made to obtain

a uniform product at the mixer and to this end grizzlies and screens removed all material below ⅜ in. to reduce segregation. As far as possible the crushed stone was used directly from the crushing plant without stock piling for when stock-piled it was found impossible entirely to prevent segregation, and as will be explained later this caused some variation in the concrete.

CEMENT

Cement in bulk was brought in box cars assigned to this service by the railway, from a mill 40 miles away.

The manufacture and shipment of the cement were under the supervision of a resident inspector at the mill. Cement, produced well in advance of requirements, was stored in bins allotted to the job. As the bins were being filled, half-hourly samples were combined into samples representing about 500 bbl., and forwarded to the Toronto laboratory for test. Cement was released from a bin only after 7-day tests were completed. Check samples from each car as it was shipped were also sent to Toronto for test. Cement used met the specifications of the Canadian Engineering Standards Association—essentially the same as those of the American Society for Testing Materials.

MIXING PLANT

The mixing plant, centrally located, was conventional in design—twin 2-yd. tilting type mixers; all materials proportioned by weighing; sand and stone weighed individually on a single multiple beam scale which could be set in 5-lb. increments; cement weighed on a separate scale accurate to 2 lb.; the water weighed in a tank on a third scale adjustable to the nearest pound.

Each batch was mixed for a minimum of two minutes, as timed by the operator on a specially-designed timing clock. In the field laboratory, two recording wattmeters, connected through a current transformer to one phase of the mixer motors, gave a graphical record of the day's operations. The meters were discarded power plant equipment, adapted to this work at little cost, and no attempt was made to get accurate power readings as the relative changes in power recorded were sufficient to register a batch, and by comparison, fractional batches and grout. By this means were obtained a permanent record of the day's output in batches, the mixing time and the duration and frequency of any delays. It has been found both on this and on other jobs, that such an installation has a beneficial effect on plant operation, for when the operating personnel know that a mechanical record is being kept, it tends to increase plant efficiency and insure full mixing

time for each batch, without continual checking on the part of the inspectors.

FIG. 1—COMPLETED HEADWORKS—NO CONSTRUCTION JOINT, BASE TO TOP, 60 FEET

FIG. 2—CEMENT STORAGE, MIXING PLANT, CRUSHING PLANT AND AGGREGATE STOCK-PILES

TABLE 2—TESTS CLASSIFIED ACCORDING TO CEMENT CONTENT OF CONCRETE

Proportions Per Cubic Yard:			W/C Ratio		No. of Tests		Compressive Strength p. s. i.		Weight per cu. ft., lb.	Absorption, Per Cent
Cement, Bags	Sand, lb.	Stone, lb.	By Wt.	By Vol.	7-Day	28-Day	7-Day	28-Day		
5.25	2622	3900	.66	.99	573	1146	2353	3565	151.1	4.56
5.38	2622	3900	.69	1.04	97	194	2310	3424	151.2	4.80
5.5	2622	3900	.68	1.02	141	282	2602	3528	150	5.01
5.75	2622	3900	.66	.99	65	130	2600	3400	151	5.81
6	2622	3900	.63	.95	109	218	2817	3733	150.8	5.59
6.25	2622	3900	.59	.89	22	44	2835	3509	151.6	..

An inspector was kept at the mixing plant during all concreting operations. His duties were to see that the proper water-cement ratio was maintained and that the proportions of cement, sand and rock were kept constant. He was given an outside limit of 0.7 as the ratio of water to cement by weight, and instructed to lower that ratio whenever possible, provided consistency and workability were maintained. At hourly intervals, or more frequently if necessary, he made moisture tests of the sand, and when required, adjusted the amount of water added to the batch.

It might be mentioned here that the moisture content remained very nearly constant at about 3.56 per cent, and the variation between maximum and minimum moisture was very small, without marked affect on consistency. The chief cause of alteration in the consistency was the variation in the grading of the crushed rock. The nature of the rock was such that crushing produced a high percentage passing the ¼-in. sieve. The vibrating screen removed most of these fines, but the natural segregation due to the heavy material throwing outward from the end of the unloading belt and the finer material falling vertically, caused pockets of fines in the storage pile. These pockets would, in time, work through to the reclaiming belt and thence to the mixer bin, with the result that for several batches the consistency would be stiffened.

The inspector at the mixer was constantly on the look-out for this condition, and to counteract it added water within the specified limits. If this did not restore the required consistency he added water and cement in proper proportion until the results were satisfactory.

TABLE 3—MONTHLY SUMMARY OF CONCRETE STRENGTHS TO SHOW UNIFORMITY OF CONCRETE

Month	Number of Tests	Compressive Strength, p. s. i., 7-Day	Number of Tests	Compressive Strength, p. s. i., 28-Day	Mean Variation of Tests From Average Comp. Strength for Month at 28-Days
1930					
August	167	2620	195	3450	10.4
September	69	2350	150	3514	10.3
October	47	2500	173	3500	9.65
November	85	2135	145	3345	9.21
December	55	2473	152	3420	10.5
1931					
January	66	2850	132	3556	7.5
February	60	2698	136	3636	8.1
March	67	2316	124	3555	8.0
April	42	2857	165	3675	15.0
May	38	2702	66	3720	6.8
June	51	2675	84	3684	7.1
July	98	2922	132	3529	7.3
August	59	2870	182	3710	6.8

From this experience and from the observations of moisture content, it is believed that the average moisture content of a large stockpile of sand could be taken as constant, and that with a rock of uniform grading the consistency would be unchanging within a permissible variation.

PROPORTIONS

Two classes of concrete were specified, "A" and "B"—the former to have a compressive strength of 2500 p. s. i. and the latter 2000 p. s. i. at 28 days, when cured and tested by usual standard laboratory procedure, but the governing considerations in designing the concrete mixtures used on this work were workability during placement and durability and impermeability. The design was on this basis rather than on that of strength as the latter was known to be sufficient for the purpose of the structure if the former requirements were met.

Experimental tests were made with the aggregate selected, to determine the proportions that would most nearly fill the above requirements, with the result that it was decided to use a ratio of sand to stone of 2 to 3, a maximum water-cement ratio of 0.7 by weight (1.05 by volume), and a minimum cement content of 5¼ sacks per cu. yd. (5 U. S. sacks approximately), and while the resulting concrete had a compressive strength in excess of that specified, it was not deemed advisable to use less cement or a higher water-cement ratio in view of the requirements for durability. The proportions used were therefore 1 part of cement, 2.86 parts of sand, 4.25 parts of rock, by weight, and the compressive strength as given by tests averaged about 3500 p. s. i. at 28 days, and on the job no differentiation was made between class A and class B concrete.

It was found that the excess of cement over that required to give the exact specified strength improved the consistency and workability which were factors of major importance in transportation and placing.

DUTIES OF PLACING INSPECTOR

The duties of the placing inspectors were in most particulars similar to the usual inspectors' duties on other works. Before concreting was started in any form, the inspector saw that the rock foundation was thoroughly cleaned and all water removed. He inspected the form work for alignment, bracing, etc., and the vertical faces of adjoining concrete, if any, and saw that it was treated according to specifications. He made certain that rock surfaces and horizontal surfaces of old concrete were given a coating of neat cement brushed in, followed by a layer of grout before the regular concrete was placed, and during the

TABLE 4—CHATS FALLS ENGINEERING BOARD—INSPECTOR'S DAILY REPORT OF CONCRETE MATERIALS

PLANT PRODUCTION

Report No. 238
Date March 27, 1931

Start No. 1—6.50 A. M.
No. 2—6.50 A. M.

Finish No. 1—6 P. M.
No. 2—6 P. M.

Time of Delay No. 1—2 hr. 51 min.
No. 2— 23 min.

Net Time No. 1— 7 hr. 49 min.
No. 2—10 hr. 17 min.

Material	Received Today	Received To date	Used Today	Used To date	In Storage	Remarks
Cement	5 cars	1019				Sand 2750 lbs. gross

	Water—Lb. Max.	Water—Lb. Min.	Water—Lb. Added	Water—Lb. Gross	W/C % By Wt.	Cement	Aggregate Fine	Aggregate Coarse	Concrete Placed Batches	Concrete Placed Class	Concrete Mixed Batches	Concrete Mixed Cu. Yds.
Div. 1	520	515	518	618	.673	918	2650	3900	126	B	126	252
Div. 5	520	515	518	618	.673	918	2650	3900	251	B	251	502
							2650	Grout	2	B	G 2	2
Day's pour											756	
Previous report											192184	
Total to date											192940	

* Cylinder No. M 237
Water (added) — 520 lb.
Moisture — 3.75%
W/C by weight — 0.689
Time 9-10 A. M.

Dry

Temperature Fahrenheit

	From	To
Outside	25	46
Inside		
Water	74	122
Sand	56	120
Stone	42	48
Concrete	58	76
Cement	69	76

Delays

Mixing Plant

Mechanical	Operation	Placing	Miscel.
No. 1	7m	164m	
No. 2	20m	3m	
Total	27m	167m	

Analysis No.
Fine 847
Coarse 844
Ave Time of Mixing 2 Min. 1 Sec.
Ave. Per Cent of Moisture 3.62

No. Mixers 2
Size 2 yd.

*—Details of Test Cylinders.

Inspector—(Sgd.) J. H. Lowry.

placing he did everything in his power to assist the contractor to obtain a satisfactory job.

Each day the placing inspector made up three test cylinders from concrete taken from the chutes at the form. These cylinders were marked with the time, date and the location from which they were taken, conditions as to consistency, fine or coarse aggregate, etc.

It was his duty both in summer and in winter to see that all concrete surfaces were kept wet after placing for a minimum of seven days.

Both the placing and mixer inspectors were required to make out daily reports on which were shown all the information required by the chief inspector. Sample reports of each are given in Tables 4 and 5.

TESTING LABORATORY

The field testing laboratory was close to the mixing plant. The building 15 x 30 ft. was of frame construction lined inside with ⅜-in. gyproc board and covered outside with tongued and grooved sheeting.

TABLE 5—CHATS FALLS ENGINEERING BOARD—INSPECTOR'S DAILY REPORT

Placing Record

Report No. 48
For Dec. 19, 1930
Shift 6.30 A. M. to 6.30 P. M. Net Time 10 Hrs. 50 Min. Foreman—F. Hunter

Location Placed	Class of Concrete	Concrete		Cylinder No. 50
		No. of Batches	Cubic Yard	
Division 6	A	251	502	* Cement—940
Unit No. 3	Grout	2	2	Moisture in Sand 3.75% Water (added)—540 W/C Ratio by Wt.—0.683 Consistency—good Time 3.30—4 P. M.
Around Draft Tube El. 148 (Base) to 163				
	Total —		504	
	Previous report —		22,976	
	Total —		23,480	

Temperature Data Fahrenheit | Plant Delays | Weather

Temperature				Time of Delay	Cause	Loss in Minutes	Weather	
Outside	21	to	26					
Concrete in form	66	to	68	1.40 P. M. to 2.20 P. M.	Power off	40	Cloudy Showery	Snow Wind
Concrete arriving at form	68	to	70				Rain	Fair X
Temp. in Form 38		—	44					

Form protected with Drier Felt and Heaters.

Consistency of Concrete—Good X Wet
Fair Very wet
Dry

*—Details of Test Cylinders.

Inspector—W. R. Bilton.

Its principal feature was an unusually effective moist room which completely solved the difficult problem of storage of field cylinders met with on a job of this kind where extremely low outside temperatures are encountered. The moist room was 12 x 12 ft., 9 ft. high, built so as to be completely separated from the walls of the laboratory by an air space. Its floor was double 1-in. boarding, with tar paper between, supported on 2 x 10-in. joists, which kept it clear of the main floor and allowed circulation of air. The sides and roof were of sheet iron nailed on 2 x 4-in. studding and ceiling joist. The air in the moist room was maintained at the proper humidity by an atomizer system whereby a fine spray of water vapor was introduced at each end. The air pressure was maintained constant, and any alteration in the humidity was affected by adjusting the valve controlling the water supply to the nozzles. The fact that the moist room was completely surrounded by an air space facilitated the regulation of the heat, as the temperature of the laboratory proper was kept constant by a thermostatically controlled electric heater. In addition to the electric heater, steam coils were installed in the laboratory to insure adequate heat in the most severe weather. The approximate cost of this automatic control was about fifty dollars.

Besides the usual miscellaneous equipment used in testing aggregates and concrete, this laboratory was equipped with two recording thermometers, one for moist room and one for outside temperatures, the two recording wattmeters mentioned before, a sieving machine, and a 100-ton hydraulic testing press.

Samples of sand were taken daily from the cars as they arrived at the storage pile and at regular intervals from the conveyor belt leading to the hopper above the mixer. These samples were delivered to the laboratory where mechanical analyses were made. A colormetric test was made also on a combined sample, but as harmful organic impurities were absent from the sand from this pit, the results were invariably good.

Each day a sample of the rock produced by the crushing plant was taken and a mechanical analysis made, the sizes above the No. 4 sieve being screened by hand. Ten pieces of rock, which passed the 1½-in. sieve, but were retained on the ¾-in. sieve, were set aside daily, and at the end of each week ten representative pieces were taken from this accumulation, and were subjected to a 5-cycle sodium sulphate test for soundness.

As previously stated, cylinders of concrete were cast daily at the mixing plant and at each point where concreting was in progress. Generally three cylinders were made at each of these points. Those

cast at the mixer were kept in the mixer plant, and those cast at the scene of operations were kept under conditions similar to the concrete mass. After twenty-four hours they were taken to the laboratory and the molds removed, when they were placed in the moist room to cure until ready to be tested. On account of the size of the aggregate, 8 x 16-in. cylinders had to be used on all field tests.

Besides the field tests, 180 6 x 12-in. concrete cylinders were made up in the laboratory in connection with various experiments, and frequent absorption tests were made of different qualities of concrete used. Sections of broken test cylinders were used for this purpose. Numerous other tests such as the effect of freezing and thawing, the effect of using rock screenings, etc., on concrete, were made as the occasion demanded.

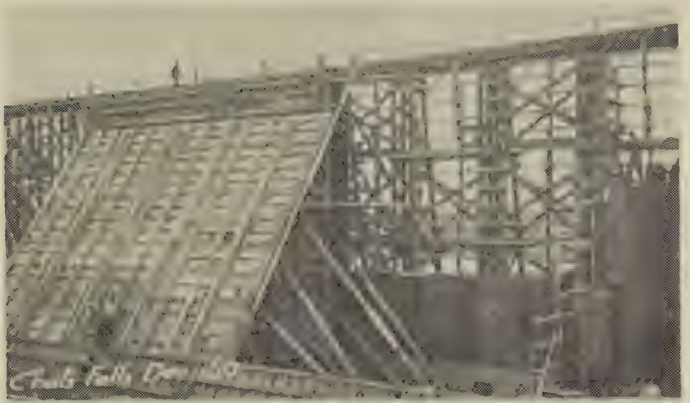
TRANSPORTATION

Transportation of the concrete from the mixing plant to the various parts of the job was accomplished in a variety of ways, but a description of this feature would be lengthy and is not essential to the purpose of the paper. Of the various means of distributing concrete only one was novel, that used to build the extreme eastern end of the dam. Here, 24-in. gauge track was laid on the decks of special 61-ft. flat cars. Ten 1-cu. yd. V-dump cars were placed on the track and the flat car spotted opposite the mixing plant where the dump cars were filled with concrete. The loaded flat car was then hauled by steam train, about 1½ miles to the earth embankment at the end of the dam. Here, the dump cars were run onto a trestle that paralleled the dam, and hauled to the point where concrete was being placed. The time in transit was about 15 minutes and on arrival at the forms, the concrete dumped easily and showed no appreciable segregation nor loss of heat.

PLACING CONCRETE IN BULKHEAD DAM

Experience has shown that probably 75 per cent of all deterioration of concrete is caused by faulty placing or poor joints and more than usual care was exercised to reduce to a minimum the likelihood of future trouble from these causes.

As mentioned in an earlier paragraph, the dam was of gravity type, of moderate height but of great length. The placing of concrete in the dam was therefore chiefly a problem of transportation, as once arrived at the form, the treatment was the same throughout. The concrete was distributed along the dam by means of a narrow gauge railway on a trestle built on the upstream side. Except in a few special instances, the dam was in 40 ft. sections with four sets of hoppers and



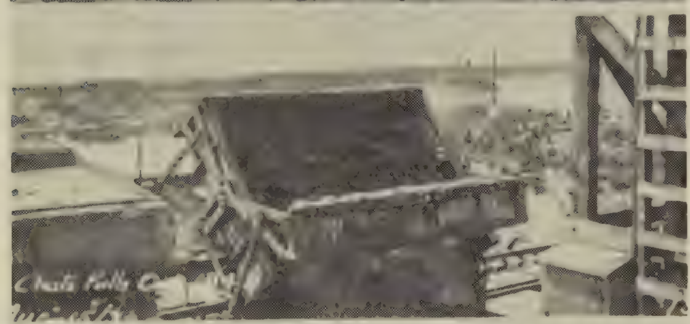


FIG. 3—PANELS USED IN FORMS FOR CONTINUOUS PLACING OF SECTIONS 50 FT. HIGH

FIG. 4—DUMPING CAR OF CONCRETE AT DISTRIBUTING TOWER

FIG. 5—TRANSPORTATION IN DUMP CARS

chutes provided at each section into which the concrete was dumped alternately. Concrete arrived at the forms in trains of from five to eight cars, and the hoppers were arranged so that two or more cars were dumped at a time. From the hopper at the top were suspended 10-in. steel sectional drop chutes which could be moved about at the lower end, thus depositing the concrete evenly over the surface.

During placing it was found that the most efficient method was to keep the concrete level longitudinally and slightly higher at the downstream face. This caused any water to accumulate along the upstream vertical face whence it could be easily removed.

A large force of carpenters was employed so that there were always sufficient forms ready to maintain continuous concreting, the principle having been adopted from the beginning that once concreting was started in a section it was continued steadily until complete, and as nearly as possible at its final resting place, every effort being made to prevent segregation in handling. Owing to the large area at the base of a section there was practically no danger to the forms from hydraulic pressure until reaching a point about 10 ft. from the top, at which time another section would be started, thus allowing the first to be completed under controlled conditions.

Each section was provided with specially designed key boxes at the bulkhead joints. When a section was built adjoining one of these previously built sections, the old concrete was given a coating of emulsified asphalt, applied with a trowel, about $\frac{3}{16}$ in. thick, and this was allowed to harden for a minimum of twelve hours. With properly designed key boxes this compound provides a water-tight joint that remains tight in all temperatures.

PLACING CONCRETE IN POWERHOUSE SECTION

Concrete was placed in the substructure from a trestle which paralleled the upstream line of the headworks piers by means of semicircular sheet iron chutes fastened on 2 x 4-in. gunwales. The chutes were supported on light trestles and were built on a slope of not less than 4 in. per foot to the downstream face of the substructure, and drop chutes similar to those used on the dam were inserted wherever necessary to carry the concrete to its place in the forms.

In placing concrete over large areas three lines of open chutes were built, and several drop chutes suspended under each. The drop chutes were spaced so that with the available radius of swing of the delivery end, concrete could be deposited from one chute to the edge of the area reached by the next chute. To deflect the flow of concrete down any particular drop chute required only the sliding out of the open chute

sections above it, and this operation could be performed by two men in about one minute. Full advantage was taken of the impact of the concrete leaving the drop chute, which caused the mass to spread without segregation by keeping the delivery end of the flexible chute as near vertical as possible. This proved to be particularly advantageous when the forms were obstructed by heavy reinforcing. Besides improving the quality of the finished concrete, this method reduced the number of men required for placing and puddling.

On this work a force of eight puddlers took care of as much as 900-cu. yd. of concrete in their shift of 11½ hours. All the puddling was done by tramping, except that in places made inaccessible by steel reinforcing bars, sticks were used.

PLACING CONCRETE IN HEADWORKS

Because of the great height of the headworks piers (60 ft.), their varying section, and the presence of intermediate and breast walls, their concreting without horizontal joints in one continuous operation presented a major problem in placing. Great care had to be used in these high piers to prevent shrinkage cracks, particularly at the junction of the bottom of cross walls with the piers, and therefore when the concrete was within 5 ft. of these junction points, the rate of placing was slowed down to 5 ft. an hour.

Contrary to expectations, adherence to a rigid time schedule did not retard the progress of the work. In actual practice it was found that two other factors automatically controlled the rate at which the level of the concrete rose in the forms. These were the conditions of placing and the area to be covered. The high forms, with a very contracted width at the gate checks, the closely spaced reinforcing steel, the form ties, and the embedded anchors for the gate guides were a formidable combination of obstacles to impede the placing; at the lower levels these conditions were partially overcome by the flow in the mass created by the impact of the incoming concrete from its long vertical drop. Four vertical chutes were installed in a pier when placing concrete, and these had to be built up in sections in place in order to thread them around the reinforcement.

WINTER CONCRETING

The weather during the winter of 1930-31, while not severe, was fairly cold, the minimum temperature averaging 17°F. The lowest temperature recorded was —22°F. The problem confronting the contractor was to deliver heated concrete at the forms, prevent this concrete being frozen during placing, and then cure it for at least 72 hours. At the outset it was decided to heat only the sand and water,

as heating crushed rock is uneconomical and the cement as received had an average temperature of over 70°F.

Careful preparations for winter work were made before the cold weather set in. A boiler plant of 450 h. p. was installed near the mixers, and the mixing plant structure was enclosed and heated by exhaust steam. A 6500-gal. tank was installed near the mixing plant as a hot-water reservoir and heater, and heat was supplied to it both by steam jets and submerged coils. The temperature of the water was controlled from the boiler room by a recording thermometer installed there. The mean temperature of the water for January was 144°F.

Special means had to be provided for handling the sand, as during transit it froze solid in the cars and could not be dumped until thawed. The contractor installed permanent steam pipes in all the sand cars in preparation for the winter. Each car was equipped with a 1½-in. header pipe running around the inside of the car about a foot below the top. Eight 1-in. pipe connections were taken off the header, four on each side of the car, the branches being about 3 ft. long, running downward and terminating in a point drawn to ½ in. diameter. In the 1½-in. main at one end of the car was a connection onto which a hose could be attached quickly for the admission of steam.

Two parallel sidings with combined capacity for 18 cars, were built adjacent to the stockpile, and a steam main was run along the tracks with outlets sufficient to serve all cars. On arrival from the sand pit, cars of sand were spotted on these sidings, the steam hose connected, and the loads covered with tarpaulins.

This arrangement thawed the sand and heated it in 10 hours to about 150°F. Cars were then dumped into the stockpile in the usual manner and the sand kept hot by steam from a main inside the conveyor tunnel which supplied a group of jets about each gate. Through all the cold weather in January 1931, the sand reached the batch hopper at an average of 105°F.

By adjusting the heat of the added water, the temperature of the concrete when discharged from the mixer could be maintained nearly constant for all outside temperatures. Where the concrete was discharged from the mixing plant into the V-dump cars, it was covered by the attendant with tarpaulins to retain its heat. The heated concrete leaving the mixer averaged 82°F. and was deposited at a distance of 500 to 1000 ft. from the mixing plant. During transportation the temperature loss was too small to be detected, and records taken on the surface of the concrete in the form, one hour later, showed the temperature still to be 67°F.

It was noticed that concrete made of heated materials was not as plastic, was more difficult to handle, and did not seem to have the life of other concrete. It was also found that as the sand was heated, its summer average moisture content increased from 3.17 to 4.37 per cent, and that the moisture was not as uniform as during the summer months. The time required to charge the sand into the mixer hopper also was increased due to the heating, but not sufficiently to affect the output.

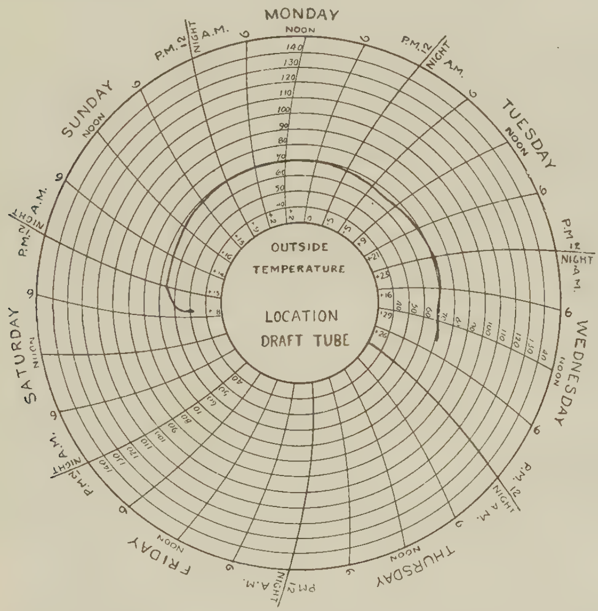


FIG. 6—SHOWS UNIFORMITY OF TEMPERATURE OF CONCRETE WHERE WINTER PROTECTION WAS USED IN DECEMBER

Protection

When preparing a form for concrete, the whole area involved was enclosed by a structure generally consisting of a strong frame covered with drier felt—a discarded material obtained from paper mills, which sells for about 15¢ per sq. yd., and is therefore more economical than tarpaulins for this purpose. Where necessary, light coverings of the same type were built to protect the men at the dumping platform.

Previous to placing concrete, all forms were heated inside to remove frost and ice from the rock foundation and reinforcing steel. To avoid trouble with live steam during the placing, dry heat was used, obtained from unit Sturtevant speed heaters, consisting of a steel casting con-

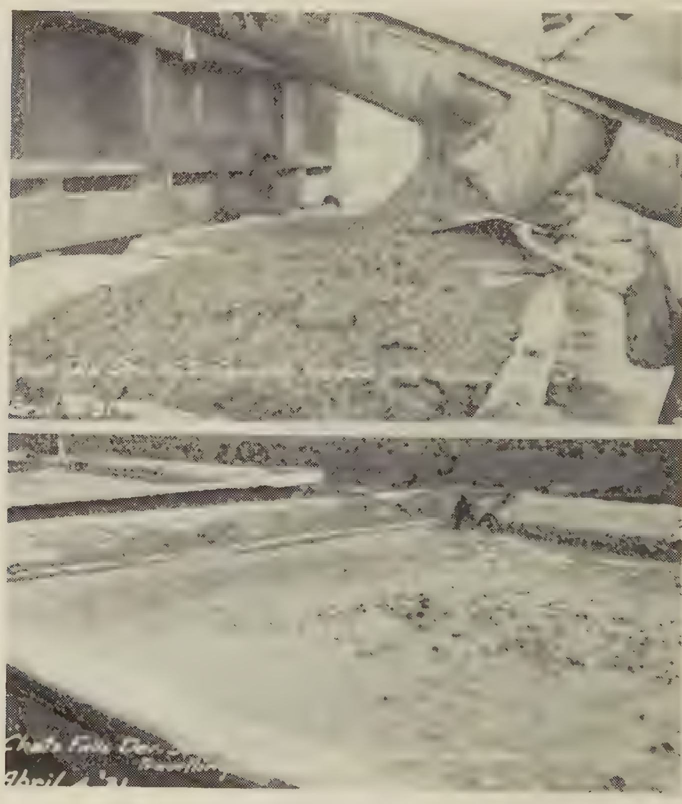


FIG. 7—(TOP) CONCRETE LOADED INTO DUMP CARS AT MIXER. (BOTTOM) SAME CONCRETE IN CAR AFTER TRANSPORTATION ON FLAT CARS 1½ MILES

taining steam coils and equipped with a motor-driven fan, which drew the air from the enclosed space and forced it over the steam coils. The exhaust from the coils was conducted through a steam hose outside the enclosure so that only dry heat was applied inside the forms. Twenty-one of these heaters were used and gave good satisfaction. The average temperature in the forms for January was approximately 42°F. while outside minimum temperature averaged 6°F.

Where open U-chutes were used during the winter, a steam line was boxed in below to keep them from freezing.

Recording thermometers were placed at the surface of the concrete under the forms to record the temperatures during the curing period and to indicate the adequacy of the protection. The chart of Fig. 6

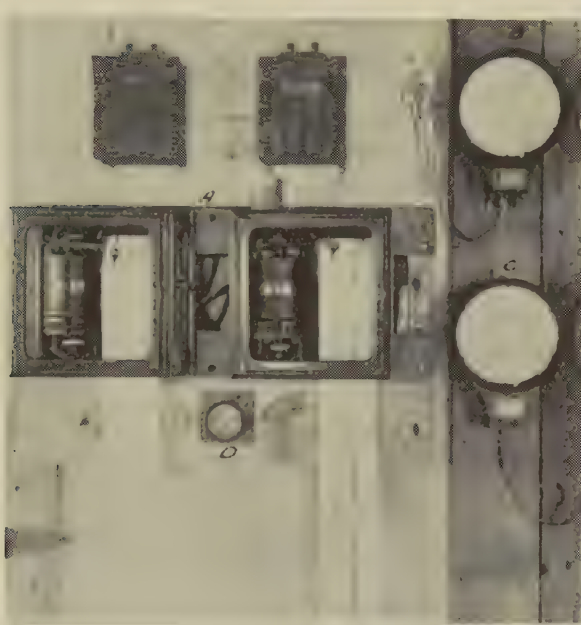


FIG. 8—FIELD LABORATORY

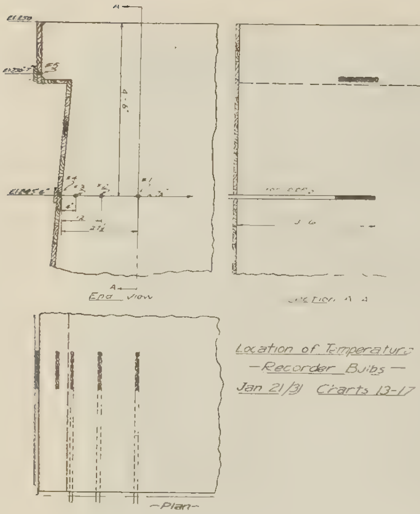
A. Batch meter

B. Recording thermometer—outside temperature

C. Recording thermometer—moist room—note uniformity of curve

D. Thermostat controlling heat for moist room

FIG. 9—HEATING SAND IN CARS



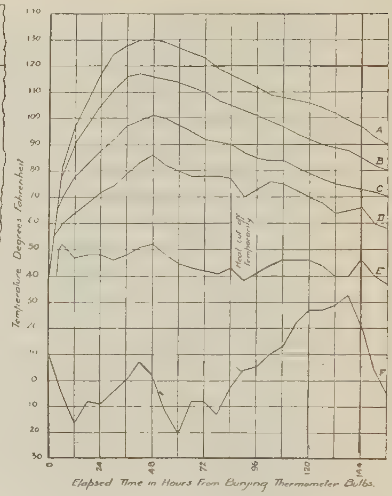


FIG. 10—LOCATION OF TEMPERATURE RECORDER BULBS

FIG. 11—TEMPERATURES DURING CURING OF CONCRETE—IN HEADWORKS, JANUARY 21 TO 27, 1931

A. 27 in. from surface
B. 12 in. from surface
C. 4 in. from surface
D. at surface under forms
E. at surface under forms at corner
F. outside temperature

illustrates how constant were the temperatures obtained in the concrete of a typical section during curing.

Reasoning from the fact that concrete generates heat in setting and that high temperatures result in the interior of large masses, many engineers feel that in such cases sufficient protection is afforded the concrete by the forms alone or by a simple covering of the top with straw or tarpaulins.

To be effective, this internal heat must be sufficient to keep the surfaces warm and cure them properly, for regardless of how good the interior concrete may be, it is the exposed surfaces that should later resist the weather.

Winter protection is costly, and so a series of field experiments was carried out to learn to what extent the heat generated in the concrete could be utilized for protection. Recording thermometers were placed immediately under the forms in various parts of the work, and at different times. Fig. 10 illustrates the way in which the thermometers were placed in several of these experiments.

Twelve recording thermometers were in service throughout the winter of 1930-31, and with them many useful data were obtained on the efficiency of various protection methods, conductivity of concrete,

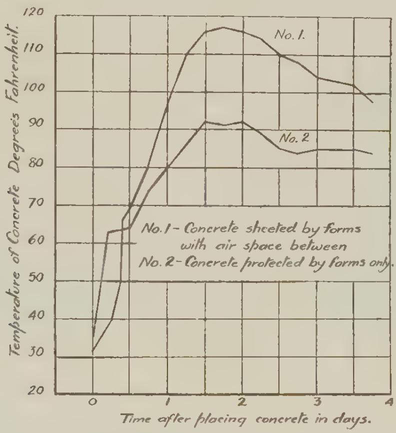


FIG. 12—EFFECT OF DEAD AIR SPACE ON CONCRETE PROTECTION

insulating value of forms, etc. Fig. 11 and 12 show sample data on temperatures in the concrete during the hardening period.

From these observations it was established that the heat generated by concrete in large masses could not safely be depended upon to protect the exposed surfaces under the climatic conditions prevailing. The ⅞-in. forms used were found to permit proper curing of flat surfaces for temperatures down to approximately 20°F., but corners and projections had to be given additional protection. For temperatures between 20°F. to 10°F., sufficient protection was afforded by providing a dead air space next to the forms. Below this temperature outside heat had to be supplied.

Curing of concrete is always essential, but it was found that the use of dry heat for frost protection made it even more imperative and a man was therefore employed constantly, day and night, wetting the finished surfaces with a hose.

When placing concrete in the headworks piers, heat was first applied inside the form to remove the frost and ice from the rock foundation and the reinforcing steel. Large curtains of drier felt were hung between the pier under construction and those on either side of it, both at the upstream and downstream face. These formed a dead air space on either side of the form, and one or two unit heaters were installed in each of the spaces. Over the top of the pier and its breast wall,

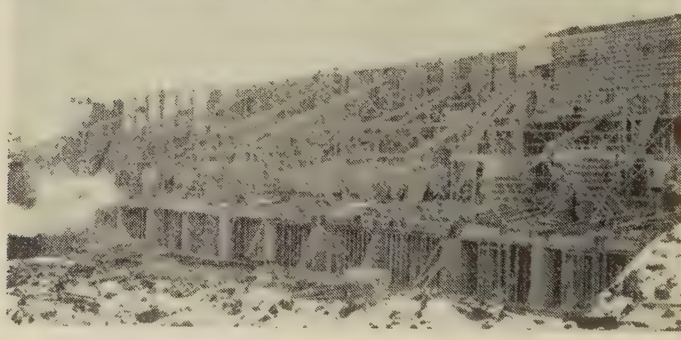


FIG. 13—WINTER CONCRETE FORMS COVERED WITH DRIER FELT

FIG. 14—CHUTING ARRANGEMENT WITH FORMS PROTECTED BY DRIER FELT AND HEAT

diaphragm wall, etc., and reaching each way beyond the next pier a housing was built of 2 x 6-in. studding carefully covered with drier felt, and heated by unit heaters. With this provision, the concrete which was deposited at a temperature of 70°F. approximately, was prevented from cooling below 40°F. at the surface in contact with the forms during the period of hardening.

Winter protection was maintained from October 23, 1930, until about the middle of April 1931, and during the entire period no time was lost through climatic conditions.

Although no precautions or care were spared to have ideal placing conditions, the additional cost of winter protection when distributed

FIG. 15—CURING FINISHED DECK OVER SLUICES

over the quantity of concrete to which protection and heating were applied, did not exceed 50¢ per cu. yd.

ARTIFICIAL COOLING OF CONCRETE

While the heat generated during the setting of the concrete can be utilized to some extent for curing during cold weather, in general, it is probably more of a detriment than a benefit to the concrete. In small structures and thin walls of concrete the heat generated is not a serious problem, but in large sections the concrete reaches a rather high temperature, around 155°F., and it takes many weeks, under ordinary conditions, for the concrete mass to return to atmospheric temperature. Some experiments were carried out on this job on the possibility of artificially cooling large masses of concrete during hardening to reduce maximum temperatures and to shorten the time by reducing the temperature of the concrete to approximately that of the surrounding atmosphere.

TABLE 6—MONTHLY SUMMARY OF AVERAGE TEMPERATURES

(Degrees Fahrenheit)

AIR, CONCRETE AND CONCRETE MATERIALS

Month	Water	Cement	Sand	Rock	Concrete	Surface Conc. in Forms	Air in Forms	Atmosphere	
								Min.	Max.
1930									
October	92.4	84.3	....	....	61.4	....	....	35.7	46.7
November	140.2	79.8	71.2	....	74.0	47.3	45.4	28.3	38.7
December	119.8	81.4	97.4	....	79.6	67.0	45.0	15.5	23.5
1931									
January	144.0	72.4	105.0	....	82.2	67.4	41.7	6.06	16.9
February	145.0	64.7	96.8	34.7	77.0	71.0	47.6	11.0	26.5
March	89.0	70.6	82.0	38.4	68.0	64.3	41.8	26.2	38.2
April	97.5	74.7	52.5	46.0	65.8	51.6	42.0	37.0	53.2
May	68.6	74.2	58.3	....	58.8	....	....	51.3	70.0
June	70.8	90.4	66.0	....	69.7	....	....	56.2	77.1

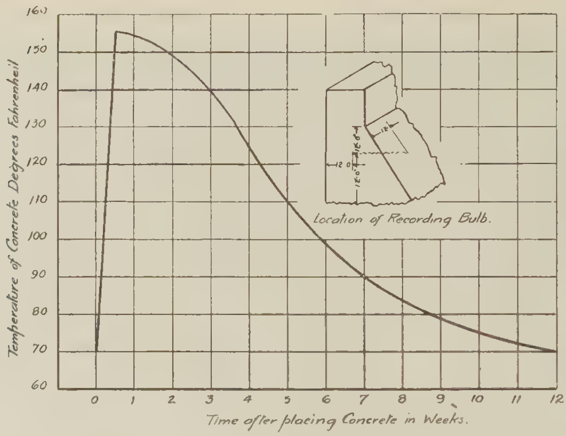


FIG. 16—HEAT GENERATED IN CENTER OF MASS CONCRETE

In the case of the concrete, Fig. 16, the peak temperature was reached in about three days, thus the greatest expansion of the concrete would be in this time, and from then on it would contract to its normal position, which might take months. These temperature changes are responsible for many cracks in mass concrete.

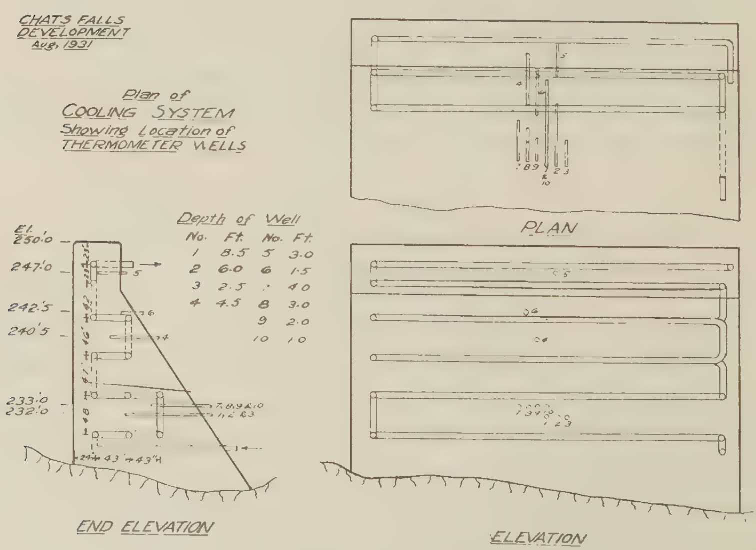


FIG. 17—PLAN OF COOLING SYSTEM

A test was made to determine the effect of cooling pipes installed in concrete. Five hundred feet of 2-in. pipe was placed in one of the bulkhead sections of the dam, and water from the lake pumped through this pipe. To record the temperatures in various parts of the concrete, ten recording thermometers were inserted in wells, placed as shown in Fig. 17. Artificial cooling was continued for 30 days, when most of the mass had cooled to approximately the atmospheric temperature. Readings were continued for more than two months.

For comparative data, thermometer wells were installed at similar elevations on the adjacent section, which was placed two days later. Comparison of the rates of cooling of the two similar sections are shown in Fig. 18 and 19.

The effect of the cooling was very noticeable after 60 hours. At this time the cooled section had reached its maximum temperature of 148°F. while the uncooled continued to rise to 155°F. In 15 days the

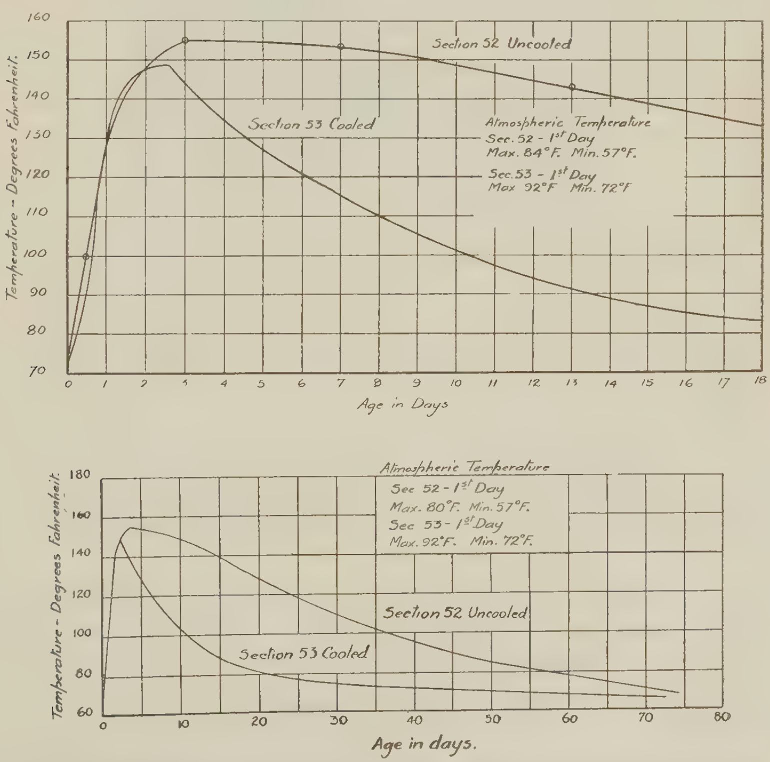


FIG. 18, 19—EFFECT OF COOLING SYSTEM ON RATE OF COOLING OF LARGE CONCRETE SECTION—JULY AND AUGUST 1931

cooled section was 87°F., the uncooled was 139°F. In 30 days the cooled section had returned to normal temperature, while the uncooled required 70 days. It may be noted that with water at the temperature used, the cooling system materially shortened the time required to return the concrete to normal temperature, but it did not materially reduce the maximum temperature.

CONCLUSIONS

Some of the points brought out on this work were:

(1) The thorough exploration of the sand pit prior to its development was a material help in obtaining a uniform supply of sand.

(2) To maintain constant consistency the fine and coarse aggregate must be constant as regards grading. This constancy is best obtained by removing fines and screenings from the coarse aggregate.

(3) The variation in the moisture content in large stock piles does not affect the consistency beyond a permissable variation.

(4) Systematic inspection at the cement mill eliminates the possibility of delays at the work due to untested cement.

(5) The weighing of all materials gives flexibility and ease of operation at the mixing plant.

(6) The addition of a separate arrangement for weighing cement at the mixing plant is a distinct advantage both as regards accuracy and speed of operation.

(7) A maximum water cement ratio with a sliding cement scale controlled the consistency and gave ample compressive strength.

(8) A field testing laboratory is an essential feature for control of concrete on a job of this magnitude.

(9) The value of the inspection and control depends largely on the thoroughness with which it is carried out, as it is necessary to have boundless enthusiasm and eternal vigilence to accomplish the best results.

(10) Concrete which was considered "dry," transported with less segregation, and handled more easily from the cars, than a concrete that was more plastic.

(11) A systematic arrangement of chutes makes placing of concrete much easier.

(12) The elimination of horizontal joints in the construction of water tight walls or dams improves the appearance, gives increased

Fig. 20—Section showing dam keys—note absence of construction joints in 50 ft. height

strength, and eliminates a source of disintegration. Horizontal joints are difficult and costly to clean, and add to the cost of placing concrete.

(13) Winter protection of concrete can be carried on with confidence, without excessive cost, even on small jobs; and with the useful information obtained on this work, more scientific planning of protection could be carried out on future work of this kind at a considerably lower cost.

(14) The internal temperature of a mass of concrete can be materially reduced during the setting period by water cooling; but more data are required on costs, and the effects of the cooling.

The general construction work was carried out by Morrow and Beatty, Ltd., Peterborough, Ont., cooperating with all parties. In this respect it is to the credit of the contractors that in spite of the many problems which they faced during the concreting period, the

entire organization gave the fullest support in all proposals advanced by the chief inspector.

The design was carried out by the engineering staff of the Hydro-Electric Power Commission of Ontario, working under the Chats Falls Engineering Board, whose members were: T. H. Hogg, Chairman; J. S. H. Wurtele, D. Stairs, and E. T. J. Brandon.

This Engineering Board, in turn, was responsible to the Chats Falls Executive Board, consisting of F. A. Gaby, Chairman; C. W. Allen, H. E. Guilfoyle, and J. B. Woodyatt.

The Engineering Board was represented in the field by H. L. Trotter, as resident engineer, and O. Holden at Toronto, as co-ordinating engineer.

Wilfred Schnarr, as chief inspector, was in direct charge of concrete control and testing.

Readers are referred to the JOURNAL *for June for discussion which may develop. Such discussion should reach the Secretary by April 1, 1933.*

Reinforced Concrete Column Investigation*

*Tentative Final Report of Committee 105***

F. E. RICHART, CHAIRMAN

In the following report, the Committee reviews the principal findings from its recent column investigation and, by way of interpretation, presents recommended design formulas for tied and spirally reinforced concrete columns.

The report has the approval of five of the seven members. Mr. R. L. Bertin and Prof. Inge Lyse approve the summary of test data but disapprove the design formulas presented.† The committee membership is as follows:

P. H. Bates	*W. S. Thomson*
R. L. Bertin	*W. F. Zabriskie*
Inge Lyse	*F. E. Richart*
F. R. McMillan	

The committee wishes to record its indebtedness to Prof. W. A. Slater, deceased October 1931, who was the original chairman of the committee, and to R. D. Snodgrass, member of the committee, 1929-31. Acknowledgment is also made of the generous assistance of many members of the Institute, who contributed to the Column Test Fund and thereby made the work of this committee possible.

1. RESUME OF PRINCIPAL TEST RESULTS

A few of the outstanding results of the recent column investigation conducted at Lehigh University and the University of Illinois under the auspices of Committee 105 will be summarized here. For the sake of brevity and simplicity, minor limitations and variations mentioned in previous test reports will not be discussed.

1. The following notation is used throughout the report:

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*For presentation and discussion at the 29th Annual Convention, Chicago, February 21-24, 1933. For first, second, third and fourth progress reports on tests in this investigation at Lehigh University and University of Illinois, see this JOURNAL for February, March (Vol. 27), and November 1931, and January 1932 (Vol. 28).

**The committee has under consideration several minor additions upon which it hopes to report finally at the time of the convention.

†A minority report by Messrs. Bertin and Lyse, here follows the majority report.

P = ultimate load on column, in pounds, applied axially at an ordinary rate of loading.

A = cross-sectional area of core (area within outer circumference of spiral) of spirally reinforced columns without shells or with light shells, or overall area of tied column (see Section 7 for spiral columns with heavy shells).

A_c = cross-sectional area of core of spirally reinforced column.

A_g = overall or gross area of column.

R = ratio, A_g/A_c.

C = a constant, for which the value from these tests was about 0.85.

f_c' = compressive strength of 6 by 12 in. concrete control cylinders.

f_y = yield point stress of vertical reinforcement.

f_s' = useful limit stress of spiral reinforcement (assumed as the stress at a unit deformation of 0.005). .002

p = ratio of cross-sectional area of vertical reinforcement to the area A.

p_g = ratio of cross-sectional area of vertical reinforcement to the gross area A_g.

p' = ratio of volume of spiral reinforcement to volume of concrete core.

k = a constant, for which 80 per cent of the test values for columns with flat ends ranged from 1.5 to 2.5 with a general average of about 2.0.

2. The tests showed that the ultimate strength of reinforced concrete columns could be expressed by a single formula of the following form:

$$\frac{P}{A} = Cf_c'\,(1-p) + f_y p + k\,f_s'\,p' \quad \ldots\ldots\ldots\ldots (1)$$

3. Equation 1 applies to columns made with a large range in amount and quality of materials, as follows:

(a) Concrete, of strengths varying from 2000 to 8000 p. s. i.

(b) Vertical reinforcing steel, of 1.5, 4.0, and 6.0 per cent, and yield point stresses ranging from 39,000 to 68,000 p. s. i.*

(c) Spiral reinforcement, of 1.2 and 2.0 per cent, and useful limit stresses ranging from 41,000 to 83,000 p. s. i.

(d) Scope of tests, included columns of varying size, age, and storage conditions.

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*Supplementing the committee's tests, later tests have been made by members of the Committee using in one case vertical steel with 96,000 p. s. i. yield point and in another case using intermediate grade vertical steel in percentages up to 17½. The results of these tests also confirm Equation 1.

4. The yield point of all columns (and the ultimate strength of tied columns) is given by the equation:

$$\frac{P}{A_g} = Cf_c' (1 - p) + f_y p_g \qquad (2)$$

5. When a column is subjected to working loads, the initial distribution of stress between vertical reinforcement and concrete follows the usual theory based upon the moduli of elasticity and the cross-sectional areas of the two materials. However, if the load is sustained for some time, the stress distribution changes very rapidly due to plastic yielding and volume change of the concrete. The effect is to increase the steel stress and to decrease the concrete stress. In test columns in dry air storage and subject to ordinary design or working loads (as determined by A. C. I. or New York City Code formulas) for 5 months, the steel stresses reached values of 30,000 to 42,000 p. s. i. in extreme cases, wherein only 1½ per cent of vertical reinforcement was present. The average increase in steel stresses during the 5-month period was about 12,000 p. s. i. in the Illinois tests and about 20,000 p. s. i. in the Lehigh tests. The average increase in steel stress in these columns from the age of 5 months to one year was only about 2000 p. s. i. more, and for 16 columns which were held another year under sustained load the increase of the 2-year stress over the one-year value averaged less than 2000 p. s. i. The increase varied inversely with percentage of vertical steel and was greater for columns in dry storage than for those in moist storage. The effect of shrinkage was much less than that of time yield. The shortening of these columns (proportional in amount to the stress in the vertical steel) due to elastic deformation and time yield of the concrete has not been shown to be of sufficient magnitude to prove harmful, with the working loads used.

6. The redistribution of stress due to time yield, and the resulting development of high steel stresses, has no effect upon the ultimate strength of a column when tested to failure. Equation 1 applies to this case.

7. While in the usual case Equation 1 gives the ultimate strength of a spirally reinforced concrete column, there are cases where the strength of the column may be even greater than this, if the shell (neglected in Equation 1) used outside the spiral is very thick or of high strength concrete. From the nature of spiral column action, the strength of the shell must be exceeded (with accompanying cracking and spalling of shell, lateral bulging and vertical shortening of column) before the spiral reinforcement can come into play. For small columns,

with relatively thick shells and made with strong concrete, the shell may add more strength than the portion (represented by the last term of Equation 1) added by the spiral reinforcement. In this case the ultimate strength is the same as the yield point of the column, given by Equation 2. This does not mean that the spiral reinforcement in such a column is of no value, since, due to the large amount of deformation required to produce failure and the initial spalling which is observable, warning will be given when a load approaches the ultimate strength. This is particularly true when the ultimate load involves bending, which will produce indications of distress in localized parts of the column. This property of "toughness" of spirally reinforced columns (similar to ductility in mild steel) deserves consideration, since a column which is capable of deforming greatly before failure will in all probability be relieved of some of its load through the stiffness of connecting members, thus avoiding complete collapse.

8. Columns of different size but made with the same percentages of reinforcement and the same quality of all materials had the same strength per unit area, aside from the effect of the shells noted above.

9. Tests of columns subject to sustained loads approaching the ultimate indicate that failure will generally occur in a few hours when the load is held at 95 per cent of the ordinary "fast-loading" ultimate. Loads of 80 to 90 per cent of the ultimate, causing longitudinal strains several times the yield point strain of the vertical steel, have been held for periods of one to two years. Such columns show extensive spalling and a gradual shortening and bending, but are still carrying the load.

2. CHOICE OF TYPE OF DESIGN FORMULA

Several factors demand consideration when the type of design formula is to be selected. Equations 1 and 2 are evidently reliable expressions for the ultimate strength and yield point of a column. Among the items that have been given careful study by the Committee are:

(a) Factor of safety with regard to ultimate strength of column.

(b) Factor of safety with regard to yield point of column.

(c) Tendency to excessive steel stresses and column shortening due to time yield and shrinkage, especially with small percentages of vertical reinforcement.

(d) The structural action of the concrete shell outside the spiral.

(e) The effect of the large deformations required to bring the spiral reinforcement into action.

(f) The adaptability of the design formula to include bending stresses.

(g) Simplicity and convenience of the design formula.

In approaching the question of new formulas for columns with spiral reinforcement and for those with lateral ties, an attempt was made to discover any difference between such columns in their behavior under load, and to determine whether different factors of safety should be used. Evidently in columns in which the strength added by the spiral is less than that of the shell, the only advantage of the spiral is in the quality of toughness it may give the column. To provide certain and unquestioned spiral action, the strength due to the spiral should exceed that of the shell. With care taken to avoid a close spacing of spirals, it seems safe to take the strength of shell concrete as about nine-tenths that of the core concrete, or as $0.75\, f_c'$.

If a spirally reinforced column be now defined as a column in which the strength added by the spiral is definitely greater, by about 15 per cent, than the strength of the shell, the ratio of spiral reinforcement required is given by the equation

$$2\, p'\, f_s'\, A_c = 1.15 \times 0.75\, f_c'\, (A_g - A_c) \text{ or}$$

$$p' = \frac{0.43\, f_c'\, (R - 1)}{f_s'} \qquad \text{(3)}$$

The amount of spiral given by Equation 3 will now be taken as a minimum requisite for spiral columns, except as noted later. It is seen that when this requirement is just fulfilled, the ultimate strength of the column is given by either Equation 1 or Equation 2 with a factor C of 0.85 and the yield point of such a column is very slightly less, due to the use of $C = 0.75$ on the shell. This makes it possible to base formulas for both spiral and tied columns on the gross or overall area. Where the amount of spiral (when used) is that determined by Formula 3, the ultimate strength may be expressed by the formula

$$P = 0.85\, f_c'\, A_c\, (1-p) + f_y p\, A_c + 0.86\, f_c'\, (A_g - A_c)$$

or for simplicity we may say,

$$\frac{P}{A_g} = 0.85\, f_c'\, (1-p_g) + f_y p_g \qquad \text{(4)}$$

The use of formulas based on gross areas not only produces consistency in design between tied and spirally reinforced columns, but is also of decided advantage when the effect of combined axial and bending stresses is to be considered. While the subject of eccentric loading is outside the scope of the present investigation and is not included in this report, the subject has been given some analytical study.

A design formula for axially loaded spirally reinforced columns is now derived by combining Equation 4 with a suitable factor of safety. In choosing the latter, some recognition is given to the tendency for high steel stresses due to time yield and shrinkage when the steel percentage is low, by the use of a slightly higher factor with low percentages. Choosing limiting values of the steel percentage (on gross area) at 0.01 and 0.08, corresponding factors of safety at these limits were taken at 3 and 2.5, with intervening values given by the expression, $F = 3.07 - 7\,p_g$. Values from Equation 4, divided by the corresponding values of F, gave design curves which were nearly linear, and which were very closely represented by the equation

$$\frac{P}{A_g} = 0.25\,f_c' + 0.45\,f_y\,p_g \qquad (5)$$

Equation 5 fits the calculated values very well, even for ranges of 2000 to 5000 p. s. i. for f_c', and 40,000 to 50,000 and higher for f_y.

The values of f_y contemplated for use in Equation 5 are minimum specification values, as, for example, 40,000 p. s. i. for intermediate grade and 50,000 p. s. i. for hard grade steel.

The values of f_s' contemplated for use in Equation 3 are 40,000 p. s. i. for intermediate grade hot rolled spirals and 60,000 p. s. i. for cold drawn wire spirals.

It is evident that Equation 5 recognizes the value of spiral reinforcement in producing toughness in a column, but does not allow any increase in load because of spiral percentages in excess of those given by Equation 3. To supplement Equation 3 in fixing a minimum percentage of spiral reinforcement, certain flat minimum percentages will also be specified to insure that proper restraint is given the vertical steel in all cases.

A design formula for axially loaded columns with lateral ties has been developed by a procedure similar to that used with Equation 5, except that a factor of safety 25 per cent greater has been used in all cases. This is equivalent to reducing the constants in Equation 5 by 20 per cent, giving the following equation for tied columns:

$$\frac{P}{A_g} = 0.2\,f_c' + 0.36\,f_y\,p_g \qquad (6)$$

3. RECOMMENDED DESIGN FORMULAS

The design formulas which the committee recommends, after a thorough analysis of the test data collected and consideration of many practical features of design and construction, are given herewith. In each case certain limitations as to design and details of reinforcement, with which the formulas are intended to be used, are given.

1. Spirally Reinforced Columns.—The maximum permissible axial load on columns reinforced with longitudinal bars and closely spaced spirals enclosing a circular core shall be that given by formula 7, using the notation given in Part 1 of this report.

$$P = A_g\,(0.25\,f_c' + 0.45\,f_y\,p_g) \quad \ldots\ldots (7)$$

The ratio, p_g, of the effective cross-sectional area of the vertical reinforcement to the gross area, A_g, of the column, shall not be less than 0.01 nor more than 0.08. The minimum number of bars shall be four. The center to center spacing of bars within the periphery of the column core shall not be less than 2½ times the diameter for round bars or 3 times the side dimension for square bars. The clear spacing between bars shall not be less than 1 in. or 1⅓ times the maximum size of the coarse aggregate used. These spacing rules apply to the bars at a lapped splice.

The spiral reinforcement shall be of such amount and quality that the load-carrying capacity of the spiral shall be 15 per cent greater than that of the concrete shell outside the core. The spiral ratio, p', to satisfy this requirement is given by the equation (see notation in Part 1)

$$p' = \frac{0.43\,f_c'\,(R - 1)}{f_s'} \quad \ldots\ldots (8)$$

In applying Equation 8, the useful limit stress of the spiral steel, f_s', shall be taken at 40,000 p. s. i. for hot rolled rod of intermediate grade (A. S. T. M. Designation A15-30) and 60,000 p. s. i. for cold drawn wire (A. S. T. M. Designation A82-27).

The spiral ratio, p', shall not be less than the value given by Equation 8, nor shall it be less in any case than 1⅛ percent for hot rolled spirals of intermediate grade or ¾ percent for cold drawn wire spirals.

The spiral reinforcement shall consist of evenly spaced continuous spirals held firmly in place and true to line by at least three vertical spacer bars. Where splices must be made in spiral rod or wire, butt-welds are recommended. The center to center spacing of the spirals shall not exceed 3 in. nor one-sixth of the core diameter. The clear spacing between spirals shall not be less than 1½ in., nor 1⅓ times the maximum size of coarse aggregate used.

2. Tied Columns.—The permissible axial load on columns reinforced with longitudinal bars and separate lateral ties is given by formula 9

$$P = A_g\,(0.2\,f_c' + 0.36\,f_y\,p_g) \quad \ldots\ldots (9)$$

The ratio, p_g, of the effective cross-sectional area of the vertical reinforcement to the gross area, A_g, of the column, shall not be less than 0.005 nor more than 0.03. The reinforcement shall consist of

not less than four bars, placed with a clear distance from the face of the column of not less than 2 in.

3. *Splices in Vertical Bars.*—Where lapped splices in the column verticals are used, the minimum amount of lap shall be as follows: (a) *For deformed bars*—with concrete having a strength of 3000 p. s. i. or above, 24 diameters of bar of intermediate grade steel and 30 diameters of bar of hard grade steel. For bars of higher yield point, the amount of lap shall be increased in proportion to the yield point stress. When the concrete strengths are less than 3000 p. s. i., the amount of lap shall be one-third greater than the values just given. (b) *For plain bars*—the minimum amount of lap shall be 25 percent greater than that specified for deformed bars.

For cases in which the requirements for bar spacing do not permit the use of lapped splices, welded splices or other positive connection shall be used.

Readers are referred to the JOURNAL *for June 1933, for discussion of the foregoing majority report of the committee, and of the following minority report. Such discussion should reach the Secretary by April 1, 1933.*

Reinforced Concrete Column Investigation

Committee 105—Minority Recommendation for Design Formula of Reinforced Concrete Columns

We, members of Committee 105, cannot accept the design formulas recommended in the majority report of the Committee, for the following reasons:

(a) Design formulas do not reflect the results of tests.

(b) The formulas do not adhere to fundamental principles established by tests or theory.

(c) The formulas are entirely empirical.

(d) They infer different factors of safety for the concrete and the steel which is brought about through an attempt to control time yield.

We therefore submit the following recommendations, relative to design formulas for consideration by the Institute.

We recognize all factors involved in the following basic formula:

$$\frac{P}{A_g} = \frac{1}{F}\left(.85f_c'(1-p) + f_y p + \frac{kf_s'}{R}\left(p_a - \frac{.85f_c'(R-1)}{kf_s'}\right)\right) \quad ..(1)$$

in which $p_a = p' + p''$ = total ratio of spiral to core area of the column

p' = ratio of effective spiral to core area

p'' = ratio of ineffective spiral to core area

the ineffective spiral being that amount required to produce a resistance equal to that of the protective shell

or $$p'' = \frac{.85f_c'(R-1)}{kf_s'} \quad \ldots\ldots(2)$$

substituting for $p_a - \dfrac{.85f_c'(R-1)}{kf_s'}$ its equivalent p'

and making $k = 2$

$F = 3.5$

equation 1 becomes

$$\frac{P}{A_g} = \frac{1}{3.5}\left(.85f_c'(1-p) + f_y p + \frac{2f_s'}{R}p'\right) \quad \ldots\ldots(3)$$

in which the amount of spiral "p_a" used in the column shall in no case be less than .005 nor more than .02, or as governed by a minimum clear spacing between wires of 1½ in.

The total unit stress shall be limited by the following:

$$\frac{2f_s'p'}{R} \leqq .4\ (.85f_c'\ (1-p) + f_y p) \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Equation 3 is recommended as a design formula for tied and spiral columns for the reason that it is derived directly from the test results and therefore adheres to fundamental principles.

The test results and investigation of structures in service have failed to reveal any ill effects from time yield and therefore we do not recognize the necessity of modifying equation 3 because of such effects.

In view of the simplicity of equation 3, we do not recognize any need for further simplification with its incidental departure from basic principles.

The basic factor of safety $F = 3.5$ applies to columns, the yield point of which coincides with their ultimate strength.

For columns in which the ultimate strength is raised above the yield point strength by the introduction of effective spiral, the yield point factor of safety is reduced in proportion to the added strength to a minimum of 2.5, as given by equation 4.

We recommend the following limitations of longitudinal reinforcement.

	Minimum	*Maximum*
Tied Columns	.005	.04
Spiral Columns	.010	.08 for reinforcing bars
Spiral Columns		.20 for structural shapes

Signed: R. L. BERTIN
INGE LYSE

Readers please note page 282 as to discussion of this report.

Supplement to Report of Committee 608:

Sliding Form Work*

BY W. R. SPROUL†

INTRODUCTION

Supplementary to Mr. Mercer's report on sliding form work, are the following data and descriptions of methods in applications of sliding forms, more especially in reference to rectangular structures. The basic principles of operation and erection are similar to those employed in erection of grain elevators. In general, our application of sliding forms has been to rectangular structures of high, clear story height and free standing single and double walls, as employed in cold storage and ice storage houses, where solid walls positively insulated and free from infiltration of air common with brick masonry, are imperative.

Our records show that no saving over stationary forms or brick construction occurs in structures less than 30 ft. high; over that height, a decided saving results, each extra foot in height showing additional saving. The initial unit cost of making and setting up forms, spread over the square feet of completed wall, is reduced with an increase in wall height.

In comparison with brick construction and slab cork insulation, our experience has shown that the sliding form method of producing concrete walls effects a saving, avoids expensive repairs and produces a completed structure in the least time.

We have erected buildings with free standing walls 30 to 100 ft. high with the floor system following after walls were completed, provision having been made in the walls to receive the floors. Methods of erecting outside curtain walls after the floor system is erected are employed in cold storage houses where the envelope system of slab cork insulation is employed.

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*Journal, Amer. Concrete Inst , January 1933; *Proceedings*, Vol. 29, p. 201.
†E. W. Sproul Co., Chicago, member Committee 608.

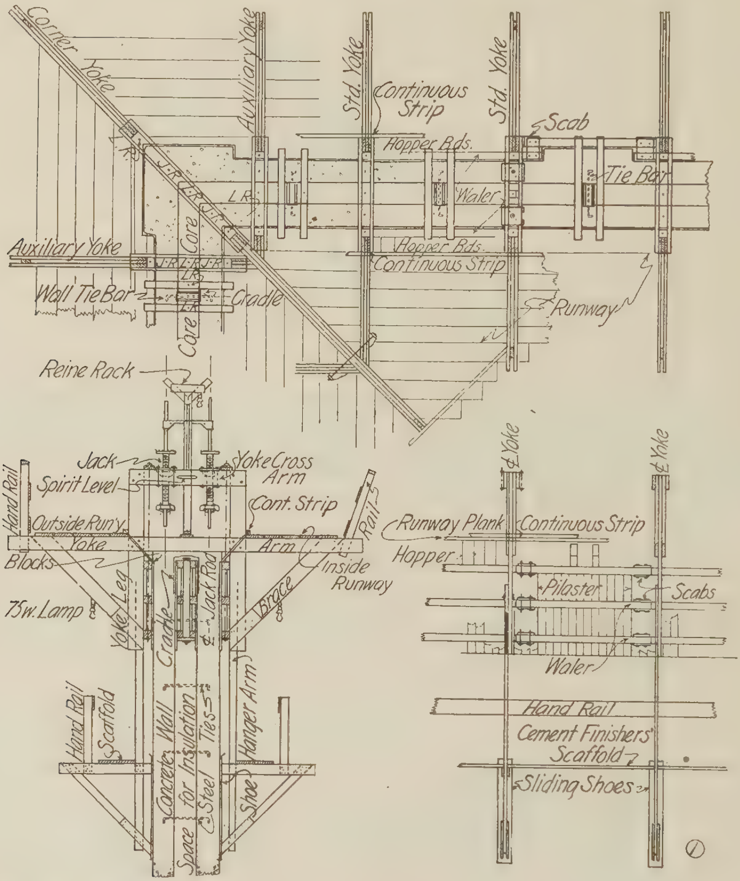


FIG. 1—GENERAL FORM LAYOUT FOR STANDARD METHOD, DOUBLE WALL SLIDING FORMS

Our forms, conforming to general practice, have panels 4 ft. 0 in. high with spacing between panel boards for expansion. A thorough application of oil is good practice and as an added precaution, we have at times, coated the panels with melted block paraffin. The theory is that in starting the pull, the congealed paraffin fills open spaces between panel boards, covers the oil surface, adheres to the concrete and slides on the oiled surface of the wood. This reduces the tendency of concrete to "freeze" to the forms. It is our practice to set panel boards or staves slightly out of plumb and to reverse the direction of this slope in adjoining panels to insure against side slip of the forms. The boards not being plumb, trowel their own joints as the form moves upward, thus reducing board marks.

Another problem, with our free standing walls, not present in elevator work, is to produce equilibrium, provide runways for concrete buggies and setting of reinforcing steel, and a platform space for jacking crews. To obtain this equilibrium, we employ several methods of straddling the wall. The yokes must also brace the walers independently, whereas in the elevator storage bins, there is the internal structure to assist in producing this equilibrium and bracing.

In single standing walls, provision must be made to maintain stability and plumbness of the walls. Outrigger jacks are employed wherein the jack rod is supported and braced. Mr. Mercer does not mention "J" posts and blocks in connection with free standing jack rods. A 10-ft. jack rod can be carried up the full height of a wall eliminating holes through beams. The same "J" post may be used as framing for beam bottoms.

In treating each method of our type of construction, it is borne in mind that the source of supply of concrete to platforms is the same as in other methods and that it is taken for granted that the plant must be adequate to supply the needs on top.

THE DOUBLE OR HOLLOW WALL

The form consists of: outside panels; inside panels; intermediate and corner cores to form the hollow space; yokes; auxiliary yokes; pilaster panels for both side and corner pilasters; cradle to seal space between cores; gates for cutting off walls; hangers for finisher's scaffold and such accessories as pockets, bucks and floor slots, as shown in Fig. 1.

The outside panels in general are made in lengths to fit between pilasters, whereas inside panels generally can be made any length, the inside face having no breaks. Our practice is to use 3 runs of walers, and experience has shown that 4 by 4-in. walers are easiest to handle. Top and bottom wood scabs bolted to the walers splice them together. In general, our practice has been to use lift rods and to bore the holes through the walers for these rods after the panels are set up, using 2 by 2-in. struts between walers to take up and distribute the strain on the walers. We have abandoned all diagonal bracing between walers, giving the forms more flexibility.

Pilaster panels are built up as a unit on the benches and ample draft allowed both in the face and at the reveals. Round corners insure ease of sliding, do not tear the concrete, and make patching easier where blemishes do occur. The larger the radius of the curve, the better. Our standards call for a 2-in. mill-made cove molding with edges same thickness as the panel boards. The walers on the pilasters

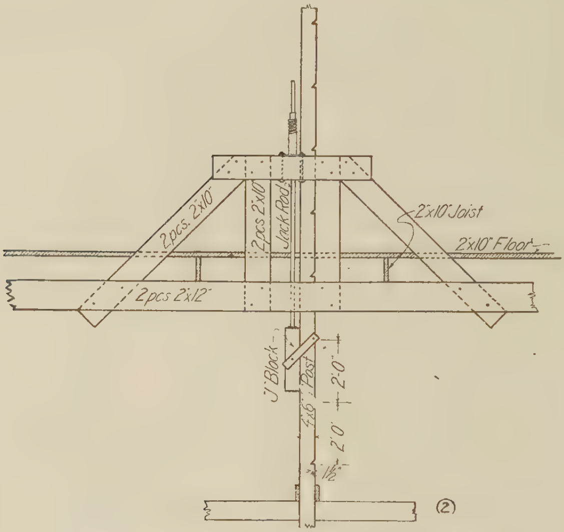


FIG. 2—OUTRIGGER YOKE METHOD

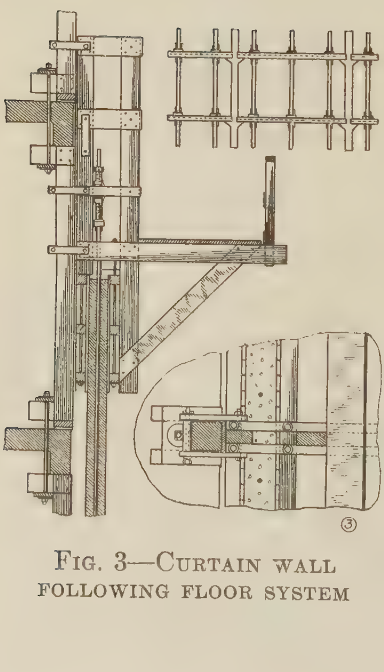


FIG. 3—CURTAIN WALL FOLLOWING FLOOR SYSTEM

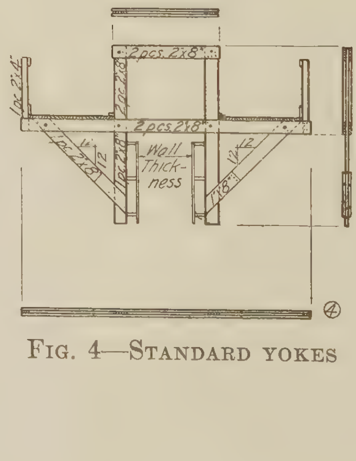


FIG. 4—STANDARD YOKES

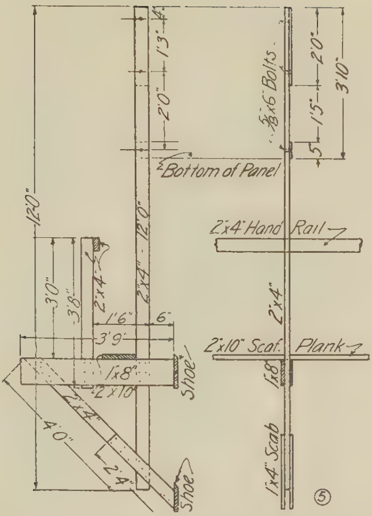


Fig. 5—Hangers for finishers' scaffold

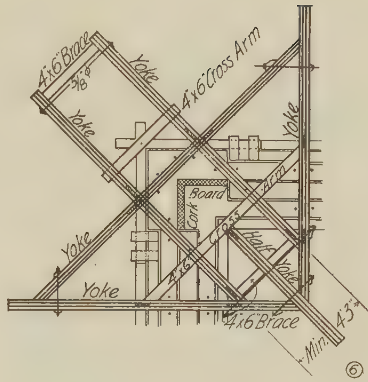


Fig. 6—Corner yokes

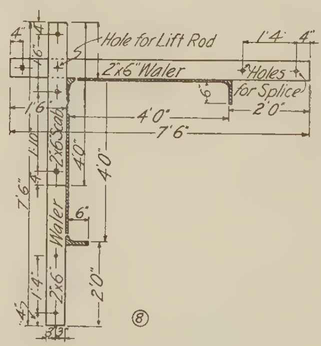


Fig. 8—Corner pilasters

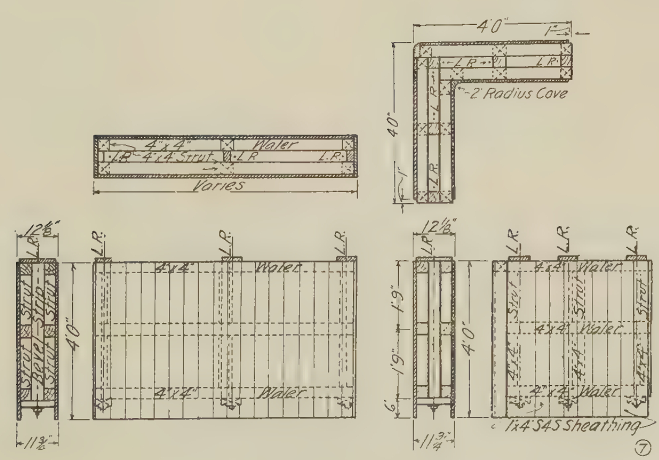


Fig. 7—Core and corner core

are allowed to extend out beyond the ends of the pilaster so they can be scabbed and bolted to the walers of the panel forms.

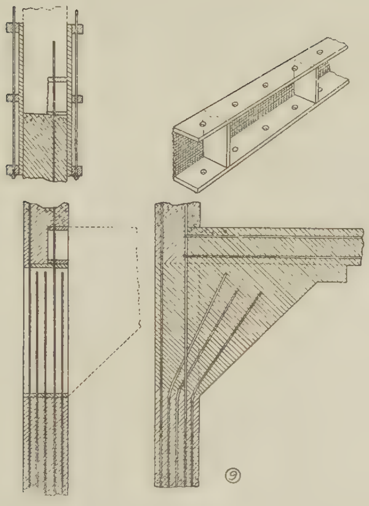
The cores consist of intermediate and corner sections, their length being determined by the spacing of yokes. As tie bars tying the walls together must pass transversely across the hollow space between walls, they must pass between the ends of core sections, and uniformity of spacing of these tie bars dictates a uniform spacing of the yokes and jack rods.

A draft to enable the form to move and free itself, must be allowed, and whereas the yokes provide this draft in the outside and inside panels, the cores must have the draft built into them. This draft is obtained by bevel strips or spacers as shown in Fig. 7. Otherwise construction is the same as for the panels. Care must be taken to seal the ends of the cores, and lift-rods and all struts between walers must be placed before setting up. The close-in piece on the top is put on after completing the forms set-up. The corner core is built very sturdily and outside and inside corners rounded by coves and rounded pieces. Extra care should be taken in making up all cores, as it is almost impossible to do any repairing after concreting has once started.

Both standard yokes, which come in between the corners, and corner yokes are used. At the corner yokes, short yoke arms, called auxiliary yokes, frame into the diagonal corner yokes. These are required to assist in jacking up corners. In our practice wood yokes are used exclusively, the material being used in or around the plant after the walls are erected. The yokes are built on templates and the legs are given a draft. The general shape gives rigidity to the frame and provides the panels with necessary bracing to resist the strains of the concrete, which of course, are greatest during initial filling of the forms. Accompanying illustrations show our standard method of straddling the wall with yoke arms extending over both, on the inside and outside to provide beams supporting the two runways and in turn a knee brace down to the walers.

Runway width where needed at the height of the handle on the concrete buggy is gained without adding additional decking, by sloping the hand rail posts outward 12 in. When the buggy is turned for dumping concrete into the form, the buggy man has ample space behind him without fear of scraping his knuckles.

The cross beams for lifting the ends of cores are 4 by 6-in. beams extending across from outside to inside walers, and are set up from the walers by means of blocks to allow clearance under cross beam similar to that allowed under the yoke cross arms. A hole is bored



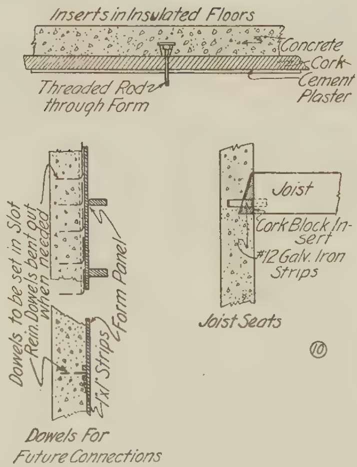


FIG. 9—FLOOR SLOT AND POCKET FOR SLAB COLUMN BRACKET
FIG. 10—JOIST SEATS

through the middle of this cross beam, through which the lifting rod is fitted.

The hangers for the finishers' scaffold are hung from the legs of the yokes. The simple type shown in Fig. 5 has been adopted after trying several designs. One advantage of this design is that by means of a single bolt the hanger is fastened to the yoke as soon as the forms are set up. It is allowed to lie flat on the ground and as the form rises, the hanger swings into position and is then securely nailed with the sliding shoe against the wall. This gives greater length to the form and tends to keep it plumb. This design also allows men working on the scaffold free passage and head room without bending or stooping. Foot boards can be brought close to the wall surface, and finally, dismantling and lowering are simplified.

In setting up forms on the foundations, it is imperative that the foundation top be as nearly level as possible. This makes plumbing the forms much easier. If the forms are out of plumb at the start, there is a constant fight all the way up, and a wave in the wall.

The cores are set up first true to line, then the panels with temporary spacers to provide the desired wall thickness. The yokes are next put in position and the walers on the panels are spliced with wood scabs and bolts. The lift rods are then set in place—a long auger bit being used to bore the holes through the walers. Struts must be

placed between the walers to distribute the strain of the lift rod. The cross beams are set after which the yokes are lined up and plumbed. Then the outside plank on each runway is spiked to the yoke arms to hold them in alignment.

The form is plumbed and leveled by means of 2 by 4-in. struts at the ends of the yoke arms. These struts are toe-nailed to the underside of the yoke arm and rest upon a mud sill board on the ground. By means of wedges under the strut, the form is plumbed and leveled.

With the form leveled and plumbed, the deck, or runway planking is securely spiked to the yoke arms; the jacks are put in position; placing hopper boards to seal the space between the runway plank and the top of the panel forms is the next step. With the steel racks, gauge sticks and light standards put up and the jack rods in place, everything is ready for concreting, with one exception—the space (about 3 in.) between the core ends which is necessary to allow for the placement of wall ties at vertical spacing as the wall progresses in height. To seal this space and prevent the concrete from filling it, a cradle is employed, composed of 2 plates straddling the core ends and extending down 18 in. from the top of the core. This still leaves open 2 ft. 6 in. of space below the cradle. This space is closed in with wood pieces, through which holes are bored to hold the initial tie bars, one placed near the bottom and another near the top of the boards. Spacers are provided between these boards extending through the spaces between the core ends. As these board fillers are temporary and are left behind when the form moves, they must not be nailed to the cores.

With the reinforcing for the first 4 ft. in place, the next step is the initial filling of concrete. To insure against lifting of this first concrete, ample dowels should extend up from the foundation. For walls which are generally 10 or 12 in. thick, 3/8-in. round bars in each face of each wall have been adopted for horizontal reinforcing with 5/8-in. round bars for vertical reinforcing between the jack rods.

Provision for receiving concrete from the tower is made by a hopper on a special set of yokes with the legs extending up to the proper height to receive a steel hopper with a quick acting gate. As the constant pressure of the concrete tends to push the form inward, the hopper is placed near but not at a corner. The forms are thoroughly soaked with water and all debris cleaned out before filling is started. As the design is such as to take the full pressure of the green concrete, it is desirable to fill the forms as quickly as possible and start the form moving upward. An additional temporary concrete source is provided to assist the initial filling.

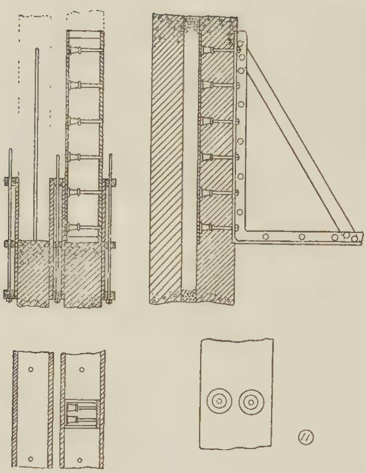
When about 3 ft. of the forms is filled, 2 men are started on the jacks. They first make a complete round, simply putting a tension on the jack rods, then they make another round turning the jacks about ¼ turn. By this time, 2 more men are added to the jacks and the 4 men then make another round with a ¼ turn. The form has now broken loose and started upward and with the forms filled, the auxiliary plant is abandoned and the jacking crew increased to its full quota of men. It is then a matter of judgment as to speed of program.

The Nelson jacks employed produce the best results with very little buckling of rods, such as occurred when using the jack with 2 outside jack rods.

After the forms start to move, it is very essential to keep them level and plumb and the usual precautions are taken to keep the jackers at a uniform pace. Working the jackers in one crew, thus picking up a section of the forms at one time, and then advancing the whole crew up to the next section produces more uniform jacking. The main jackers proceed without checking themselves as to level, and are followed by 2 or 4 men, as the speed warrants, who check for level and back up jacks. To assist the leveling crew, spirit levels are placed on every third yoke. If these are kept level, the form in its entirety will be level and plumb. To guard against the condition wherein a section of the form may be out of level, gauge rods are employed using ¾-in. conduit pipe started off the foundation and graduated with 1-ft. elevation marks. During each shift at the lunch hour, the engineer checks the elevations and levels up the forms. At intervals between these checks, the general level is kept by sighting the yoke heads.

As a free standing wall has a greater tendency to get out of plumb than a series of elevator tanks, constant watch must be kept. Corrections must not be too violent or a waving in and out of the wall will occur.

The system adopted for floor anchorage is to provide a continuous slot of the required height and depth by inserting boxes of wood without front or backs and of such lengths as can be handled easily under yoke arms and cross beams. Backs for these boxes, as shown in Fig. 9 are made of ordinary fly screen wire, which allows the air behind and a small amount of the excess water in the concrete to pass through, thus keeping the boxes in position. Dowels are driven through holes provided at 6 in. centers in the tops and bottoms of these boxes. Later, the hooked-end rods of the floor system are passed behind these vertical dowels to provide desired anchorage. When placing a continuous floor slot in the inside wall, concrete is brought to the desired level of the underside of slot, the concrete level in outside wall being



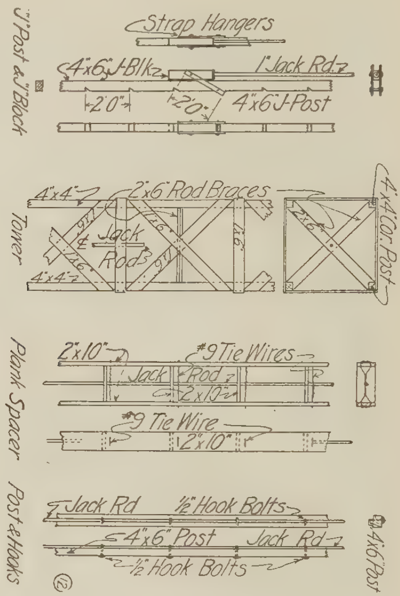


FIG. 11—GROUP INSERTS

FIG. 12—"J" POSTS AND OTHER BRACING OF FREE STANDING JACKS

allowed to get low. When the top of the panel form gets to the top elevation of the box, the form is leveled up and stopped until boxes are lined up to a uniform level. The pour continues in the outer wall and concrete will be at the same level in both walls. Concrete can be placed behind boxes and motion of forms started again. Boxes and pockets are placed in a similar manner except that no stopping or special procedure is required unless a great many are placed at the same time.

In buildings designed with a loft or attic space over insulated and refrigerated rooms, at loft floor elevation, the inside wall is stopped and only the outside or single wall is carried up the height of this attic space. It is necessary to carry up the inside forms empty. This necessitates spreaders in the inside wall space to maintain the position of the core, which still is acting on one side to form the outside or single wall. The inside jack rod must be braced to keep it from buckling, for it is still doing its work in raising the form. To brace this jack rod, one of several methods as shown in Fig. 12 may be used.

It is necessary also to resort to bridges to get a continuous wheeling platform in buildings where only 3 side walls are to be of concrete and the other side left open, and the ends of the walls left to frame into a

future unit on the open side. Where these bridges are long, towers brace jack rods which lift girders under the bridge, which in turn, lift the bridge at intervals between the walls.

Single wall construction is similar to that of double, except that to have control over equilibrium and maintain a stable structure, an outrigger jack is required on each side of the wall at every third regular yoke. These outrigger jack rods require bracing to insure against buckling. One of several methods for the loft floor single wall may be employed. These outriggers and towers also come into play when it is necessary to omit one side of the yoke, a condition which arises when the new structure adjoins an existing structure built on the property line.

Another design of form is used to produce the same double-wall result, by means of platform on inside only, employing outrigger jacks and yokes around the inside of the building to carry the platform, or runway. This method has certain advantages, but requires constant bracing and additional framing for outriggers. The advantages are that wheeling can be carried on in any direction, means are provided for buggies to pass, a more stable deck is produced, it is easier to control equilibrium, and concrete pouring and placing of reinforcing are simplified. Otherwise, the general procedure is the same.

Supplementary to (and applicable to both of the systems described), pilasters are formed inside the hollow space between the walls by making the distance between core ends equal to the width of the pilaster and producing the desired reveal, by filling the balance of the space with corkboard as the wall rises. By this method, the corner core is omitted and added strength of the pilaster is obtained. It makes possible future openings without disturbing the regranulated cork insulation in adjoining pockets, and fire starting in one of the pockets would be confined to that pocket.

Pouring curtain walls after the floor framing system of a building has been erected is applicable to that type of cold storage house which uses a continuous corkboard insulation in the walls, with floor insulation meeting the wall insulation, forming a complete envelope of insulation for each floor with no conductor between.

The condition of insulation demands that a space be allowed between the floor ends and the wall to provide a space for the wall insulation. The problem then is to slide up through the space at the edges of the floor and bring into existence a wall 6 or 8 in. away from the floor as desired. To accomplish this, a yoke is used similar to half a standard outside yoke. The inside panel slides on a guide post securely anchored

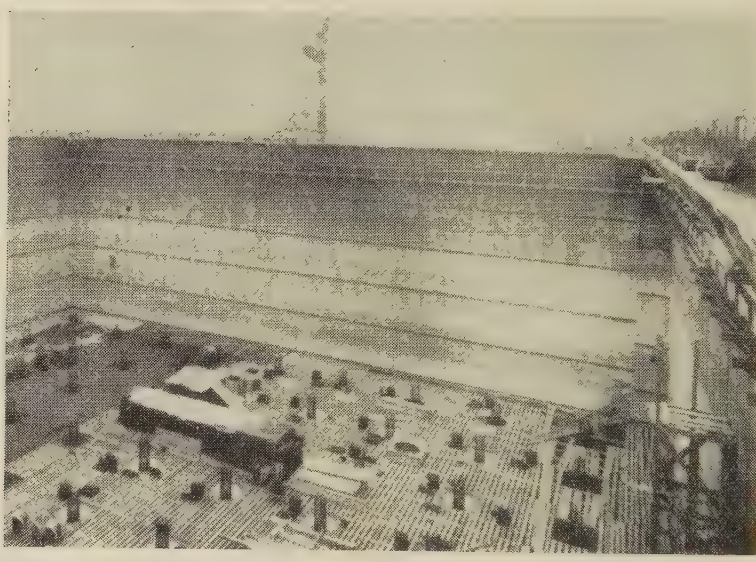


FIG. 13—CITY ICE AND FUEL CO. PLANT, CLEVELAND, OHIO

to the floor system, and equilibrium is secured by extended yoke legs, guide rollers and clamps. Provision is made for at least 3 of 4 clamps in operation with one only being disengaged when passing the floor system between guide posts. It is necessary to have guide posts for 3 floors, reusing those from below as operation proceeds. The plant used for the floor system may be used for placing concrete and the floors themselves used for runways, with chutes to deposit the concrete, either directly to the forms or to buggies on the outside platform.

This method of operation need not be confined to cold storage houses, but can be used on any type of structure for curtain walls if given the space of 6 in. in which to operate between floor ends and the inside face of the curtain wall. The space outside and adjoining the walls, however, must be free of any obstructions or adjoining structure for a distance of at least 5 ft.

FINISHING

As a rule, the inside wall needs no dressing, as it is generally clean and free from blemishes, presenting a uniform appearance. The outside wall shows the blemishes, but these can be reduced to a minimum by keeping the forms clean ahead of the pour and by maintaining proper slump of the concrete. A 3 per cent admixture of Celite has been found very beneficial.

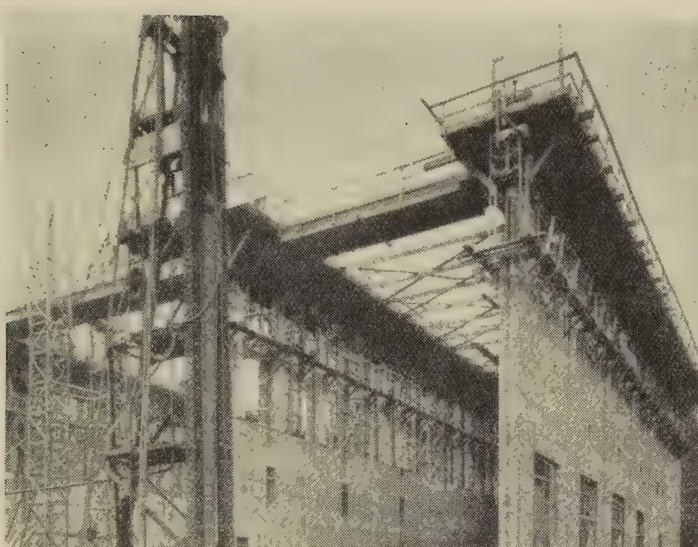


FIG. 14—CITY ICE AND FUEL CO. PLANT, PITTSBURGH, PA.

Only the foreman of the finishers carries a trowel, and no "mud" is applied unless he specifically orders it. The other finishers simply brush down the wall with a brush and clear water, which is all that is necessary to seal up the minor blemishes and board marks.

WINTER WORK

Most of our work is carried on in the winter, and we protect it by using hot water and heated aggregates. Salamanders are hung from the finishers' scaffold and smudge fires are built around the base of walls. The space between walls is sufficiently warm, due to the chemical action of the cement. In very severe, sub-zero weather tarpaulins house the decks and finishers' scaffold.

A structure poured in the winter with the proper precaution produces a better concrete wall than one poured in the heat of summer, and is less likely to show shrinkage and hair cracks as the shrinkage takes place in the early stages of hardening.

Protection against heat is as essential as against cold, and the platform is equipped with a means of spraying the finished wall with water to guard against too rapid dehydration.

FEDERAL COLD STORAGE CO. PLANT AT CLAYTON, DELAWARE

This was one of our early installations and the problems, such as the single wall of loft story, received much thought. Expensive bracing

was employed, since found unnecessary. On this job, which was poured during Christmas week, we provided 2-in. steam lines under the core. This also, later was found unnecessary.

CITY ICE AND FUEL COMPANY PLANT AT NILES

As usual, with an icing station, the season storage and daily storage houses are of different heights, and the forms are pulled simultaneously. When the top of the daily storage is reached, it is cut loose and only the season storage forms carried up. To make this cut-off, gates or panels are provided and at the proper time are inserted into the slots to receive them. Thus the concrete is held from going into the daily storage wall. The form itself is cut through at the walers and stays behind.

On this job, openings were provided for future extension of the season storage house. These openings were made, then sealed, the sealing process having 4 operations. By means of traveler scaffolds down from the finishers' scaffold, the 4 operations were performed as the work progressed.

CITY ICE AND FUEL COMPANY PLANT AT CLEVELAND

This plant consisted of a 6-story and basement cold storage house 200 by 200 ft., and an ice manufacturing room and an ice storage room for retail trade. On this job the group insert design, Fig. 11, was employed for anchoring canopy trusses. The grouping feature insures positive, correct relative spacing of inserts and eliminates looking for them and also puts the full thickness of the wall to work carrying the load as the threaded spud rod is removed and the permanent bolt inserted in the hole thus formed.

Screen wire back for floor slot boxes was also employed with success. The pocket and system of reinforcing dowels to produce a wall bracket or wall capital to take care of the negative movement occurring at the wall in a flat slab design was devised for use on this job but not used, as the engineers for the City of Cleveland did not consider wall capitals necessary.

CITY ICE AND FUEL COMPANY PLANT AT PITTSBURGH

This plant also consisted of ice manufacturing room, ice storage and cold storage house 200 x 197 ft., 7-stories high.

Complications due to physical conditions of the site were many. The 16th St. viaduct leading to the bridge over the Allegheny river ran at an angle to the building line and interfered with yoke ends for a section of the east wall of the ice house. It was necessary to design the yoke ends so that a portion could be removed in passing the bridge,

then, as it was advisable to have the full runway, they were built out to standard length when the bridge rail was passed.

The ice house and ice manufacturing room was built independently of the cold storage house, but the east wall of the cold storage house later was to close in the west end of the ice house and manufacturing room. The structure had an open end, over which was built a bridge to provide a continuous runway for the concrete buggies. The ice manufacturing room was on the second story over the ice storage room, and also over the engine or compressor room. The walls of the ice storage room were double for a height of 30 ft., at which point the center wall, or south wall of the storage room was stopped. The north and east walls of the storage were carried up 27 ft. as single walls, together with the engine room walls, which were single walls, from the foundations up the full height of 57 ft.

It was decided to tie and brace the center wall forms to the other wall forms and also to carry up the 2 bridges the full height of 57 ft. When the 30-ft. elevation was poured, the center wall and the bridge ends were carried up on free standing jack rods, braced with false work. The operation was successful but caused a constant worry.

Adjoining the cold storage on the front half of the east elevation was a residence on the building line for a height of 60 ft. Panel forms with flattened walers were placed, but the pilaster face was on the line. Guide strips fastened to the adjoining walls with pipe rollers on the panel walers and outrigger jacks on the inside were used to maintain equilibrium until the adjoining building was passed, when standard size yokes were built on the east side producing a uniform face of wall and pilasters above.

SOUTHERN ICE AND UTILITIES COMPANY PLANT AT NASHVILLE

On this job a distance of 60 ft. between walls was bridged with three latticed trusses using 16 ft. plank for bridge deck. The bridge was carried entirely on the forms with no center support and caused no trouble.

Readers are referred to the Journal *for June 1933 for discussion of this and the preceding report (January* Journal*). Such discussion should reach the Secretary by April 1, 1933.*

Discussion of a Paper by Gemeny and McCullough:

"The Freyssinet Method of Arch Construction Applied to the Rogue River Bridge in Oregon"*

Charles S. Whitney†—The authors' investigation of the Rogue river bridge is one of the most interesting studies of the behavior of concrete arches during construction which has ever been attempted. The care with which the work was planned and executed and the importance of the problem involved would seem to justify the publication of the resulting data in great detail, that definite conclusions may be adequately supported. It also seems important that the observations be continued over a period of years so that the effect of shrinkage and plastic flow and temperature changes may be determined as nearly as possible.

The authors state that the paper is in the nature of a preliminary report and that considerably more study must be given to the data before complete conclusions can be drawn. They do, however, state tentative conclusions and the writer wishes to point out that the data presented in the paper may be subject to interpretation which does not support some of these tentative conclusions. The published data do not appear to lead definitely to the broad conclusion that the Freyssinet method of arch construction is "effective in eliminating the deleterious effects of rib shortening due to all causes and permits an adjustment of the arch to its most favorable elastic state under combined dead and live loads" and that "this method permits greater economy in arch construction even in moderate span lengths." Although the authors' published‡ figures indicate economy in the Rogue river bridge, the writer questions the basic assumptions used in computing the stresses in the arch with and without the Freyssinet adjustment.

In view of the incompleteness of the data presented, the writer hopes that the authors will clarify their discussion.

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*Journal, Amer. Concrete Inst., October 1932, *Proceedings*, Vol. 29, p. 57.
†Consulting Engineer, Milwaukee, Wisc.
‡"Designing the first Freyssinet arch to be built in the United States," by C. B. McCullough and A. L. Gemeny, *Eng. News-Record*, November 26, 1931, p. 841.

On page 78 is the statement: "This seems to indicate that the telemeters recorded in this case only elastic strain." Is the reader to infer that the telemeter can in any case differentiate between shrinkage, elastic, and plastic strains or that the correspondence between the telemeter readings and the computed elastic strains was due to the absence of other than elastic strain? It is obvious that the telemeter must record any strain, whether it is due to shrinkage, plastic flow or elastic deformation, and that the stresses cannot be obtained by multiplying the telemeter readings by the modulus of elasticity. The telemeters record stress changes only when the period is so short that the effect of shrinkage and flow are negligible.

The stresses shown by Fig. 12 to 21 inclusive are, therefore, not actual. This is shown by Fig. 20, indicating an increase in stress at point 708 between readings 109 and 110 when the rib rested on the rings for 20 hours. In this time the thrust evidently decreased by an undetermined amount as the authors say that the jack pressure was brought back to 1,040,000 lbs. at 111, the same as at 109. The strain recorded by the telemeters during this 20 hour interval was largely plastic and not elastic, and has no significance when multiplied by the modulus of elasticity. This is true to a varying degree of all of the telemeter readings.

During the 20 hours when the rib rested on the rings there was a plastic shortening of the rib, as indicated by the telemeter readings. Fig. 10 indicates no vertical movement of the crown during this time which would naturally result from rib shortening. The explanation is obviously that the rib has changed shape sufficiently to compensate the crown deflection. The change in shape is indicated by rotation observed at various sections. The crown opening between readings 109 and 111 is therefore not a measure of the inelastic rib shortening as stated by the authors (page 68) because, although the crown was at practically the same elevation, the shape of the rib was changed. It is interesting to note that the change in strains shown at the different sections by the telemeters indicates a total rib shortening of considerably less than the 0.12 in. crown opening between 109 and 111.

The accuracy of the plastic shortening measured for spans 1 and 2 by lifting the arch to the highest point and again placing it on the forms may be questioned because of the effect of the friction of the forms and the elasticity of the falsework.

Page 68 the authors say "In Fig. 11, at zero deflection, when the crown was at its original elevation on the centers, it is observed that the maximum crown spread is 0.70 in. . . . This shortening of

0.70 in. included elastic shortening, plastic shortening and shrinkage which had been restrained by the reinforcement and centers." They ascribe the greater part of this 0.70 in. shortening (0.38 in.) to "delayed shrinkage." The writer questions both the amount of the rib shortening and the nature of the shrinkage effect.

It is not clear why it should be assumed that the rib shortening should be represented by the crown opening when the crown elevation was at its original value as at that time every other point of the rib except the springings was evidently at a different elevation from the original and all points measured showed material rotation. The crown opening could therefore not be an accurate measure of the rib shortening.

Page 78 the authors state "Apparently the major part of the shortening is due to shrinkage which was restrained by the reinforcement and the forms. The reinforcing steel is put in compression and the concrete in tension which produces minute cracks at intervals along the rib. As the jack pressures were applied, these cracks were closed. This movement would be reflected in the telemeter readings only in the case where the telemeter crossed a crack. Obviously, in this span none of the telemeters was across a crack because it would have been observed in the preliminary readings before jacking."

The authors have produced no tangible evidence of the existence of these shrinkage cracks, and other investigations indicate that they do not exist. The reinforcement in the ribs averaged about 1 per cent, the concrete had a strength of about 5000 p. s. i. and the shrinkage at the time of jacking was about 0.00015 in. per in. In reporting on recent tests of concrete columns made at the Univ. of Illinois, Richart and Staehle* state: "In the case of the columns held for a year under no load, however, no shrinkage cracks were observed, even in columns with 6 per cent of vertical reinforcement. Since the difference between the strains in plain and reinforced columns was in excess of 0.0002, an amount greater than the ultimate tensile deformation for concrete under part loading, it seems that there must have developed simultaneously with the shrinkage a considerable tensile plastic flow, which inhibited the formation of cracks."

The failure to find cracks in columns with 6 per cent reinforcement would indicate that they would not influence the behavior of an arch rib with about one per cent of reinforcement. The restraint of reinforcement and forms does produce a shrinkage stress in the concrete but at the earlier ages this stress is also materially reduced by plastic

*"Fourth Progress Report on Column Tests made at the University of Illinois," JOURNAL Amer. Concrete Inst., January 1932, *Proceedings*, Vol. 28, p. 284.

flow. The writer is inclined to believe that the delayed shrinkage effect as described by the authors does not exist to any material extent. The observed strains were too small to have induced stresses in the steel high enough to have cracked the concrete.

In computing the stresses from the telemeter readings, the authors used a value of E equal to 4,000,000 p. s. i. The average value of E for specimens 52 to 83 listed in Table 1 is 4,630,000 p. s. i. at ages from 76 to 98 days. In the specimens tested at ages of from 14 to 30 days, the average is 4,070,000 p. s. i. It seems possible that the value for the massive rib might have been materially higher than 4,000,000 and that the difference in the key sections may have effected the behavior of the ribs to some extent.

The writer can find nothing inconsistent between the data obtained from the Rogue river bridge tests and the theory of stress release through plastic flow.†

When a load is placed on an arch rib there results an immediate elastic deformation. The resulting strains are at first elastic but plastic flow starts at once and gradually produces plastic strains which may result in increased total strains or in a transfer of elastic strain to plastic strain depending upon conditions. In restrained arches, there is a tendency for permanent strains to change from elastic to plastic with a resulting reduction in stress. Telemeters unfortunately cannot differentiate between elastic and non-elastic strains.

A thorough study of the results of the observations on the Rogue river bridge will help to solve the problem of arch behavior but it is doubtful if the tentative conclusions of the authors will be verified.

†Progress Report of Committee 312, JOURNAL, Amer. Concrete Inst., March 1932, *Proceedings*, Vol. 28, p. 479.

Readers are referred to the JOURNAL *for June, 1933, this volume for conclusion of this discussion.*

Mass Concrete Research For Hoover Dam*

BY BYRAM W. STEELE†

INTRODUCTION

THE design and construction of Hoover Dam has necessitated the solution of problems of greater variety and magnitude than any similar project ever built. Those pertaining to mass concrete construction occupy an important position.

In the last 20 years the immense amount of research work on plain and reinforced concrete has been largely confined to investigations of concrete in which the maximum size of aggregate was about 1½ in., rather than to mass concrete containing aggregate of all sizes up to and including cobbles. Past research, while applicable to a large percentage of all concrete work, does not furnish adequate data for the solution of mass concrete problems such as will arise in the construction of Hoover Dam. While numerous mass concrete structures have been built regardless of this lack of information, the magnitude of the undertaking, which involves more concrete than has been placed by the bureau since its organization in 1902, seemed to warrant an investigation to secure the fundamental data required for the solution of these problems.

SCOPE OF INVESTIGATIONS

The program of investigations as originally outlined embodied such subjects as volume change, foundation and contraction joint grouting, portland cement, mass concrete strength and mix proportions, elastic properties of mass concrete, thermal properties of mass concrete, permeability, precooling concrete ingredients versus concrete refrigeration, action under axial, biaxial and triaxial loads, bond and sliding friction, shear, uplift, and durability.

In the beginning, certain tests were assigned to different organizations, but as the work progressed, some of the tests originally contemplated were dropped and others were postponed or transferred from one organization to another. The investigation of portland

*Presented at the 29th Annual Convention, Chicago, Feb. 21-23, 1933, as the first of series of eight papers on Hoover Dam research.
†Bureau of Reclamation, Denver.

cement started by the Bureau of Standards in Washington, was continued and greatly expanded by the University of California under cooperative arrangement with the Bureau of Reclamation, and will be concluded, insofar as Hoover Dam is affected, by what are termed "comparison of cements" tests on mass concrete now in progress at the Bureau of Reclamation laboratories in Denver.

The mixing of mass concrete was recently studied at Owyhee Dam during its construction. These studies are being continued at the Denver laboratories and at the concrete mixing plant at Hoover Dam. The studies on strength, thermal and elastic properties and permeability are now in progress in the laboratories in Denver. Tests of stress under axial, biaxial and triaxial loads remain to be considered. Some work has been done in Denver on shear, bond and sliding friction, and uplift. Volume change has received both field and laboratory consideration by the bureau. Durability is being studied at the University of California.

Thus it will be seen that the field covered by these investigations is fairly comprehensive. However, it should be pointed out that the Bureau of Reclamation has undertaken only such research as applies directly to concrete design and construction in structures of the type of Hoover Dam and related works, and particularly the former inasmuch as mass concrete conditions furnish the outstanding problems. Considerable pioneering in technique and development of testing apparatus have been necessary since much of the research is outside the range of standardized laboratory practice. More than a year was spent in preliminary testing in order that the final data would not be tainted with tests of the apparatus, but would represent a true story of the performance of the test specimen. The major subjects of the investigation will now be discussed in the order of their relative importance.

VOLUME CHANGE

Since volume change occurs in all concrete work, it is common practice to place contraction joints, or designed cracks at appropriate intervals to improve the appearance of the structure and to prevent cracking at undesirable locations. In a dam, either the contraction joints must not open or the opening must be sufficient to permit a satisfactory job of grouting if a monolithic structure is to be obtained.

Foremost of the mass concrete problems is the control of this volume change. But back of all this is the underlying reason for desiring to control this change. As the dam shrinks, due to loss of heat, the particles which make up the cantilever elements are held in intimate

contact by gravity and consequently when the water load is applied the minimum of deformation takes place. With the arch elements, however, the condition is entirely different. As the dam shrinks gravity does not assist in holding the arch element particles in intimate contact and as a result the vertical contraction joints open up and arch action is prevented until they are again closed. This closing can be accomplished in two ways; (1) by deflection of the cantilever elements downstream in a radial direction until the arch voussoirs again come into contact, or (2) by grouting the contraction joints so that the water load will be distributed to both the arch and cantilever elements with the minimum of deflection and in the proportions assumed in the design. Obviously the latter is preferable, and it is the principal reason for considering accelerated shrinkage of the concrete in the dam and grouting of the contraction joints.

Shrinkage in a massive concrete dam is normally distributed over a long period of years and is due principally to two causes: (1) the loss of chemical heat generated during the hardening process, and (2) the loss of initial heat, which is the heat due to temperature in excess of mean annual at the time the concrete is placed. The impracticability of regulating the rate of heat dissipation was the controlling factor in determining that the concrete in Hoover Dam would need to be artificially cooled if the contraction joints were to be grouted before the dam was subjected to water load. Were it economically practicable to construct Hoover Dam at a rate such as to allow all, or nearly all, of the heat to dissipate under natural conditions during the construction period; or, to construct it only during the cooler parts of the year when the difference between the initial concrete temperature and mean annual was equal to the temperature rise due to the heat of hardening, then all problems of future volume change of any moment would disappear, since the final temperature of the concrete would be at or below mean annual and no shrinkage would occur. Such a condition would obtain if for example the temperature of the concrete at the time of placing was 40°, and the temperature rise due to generation of heat during the hardening process was 32°, making the resulting temperature in the concrete 72°, which is approximately mean annual for that locality.

To obtain data on volume change in mass concrete under actual working conditions in the field, a test was made during the construction of Owyhee Dam, in which a pipe cooling system similar to that proposed for Hoover Dam was embedded in two adjacent panels or blocks near the top of the dam. This experimental section was roughly 100

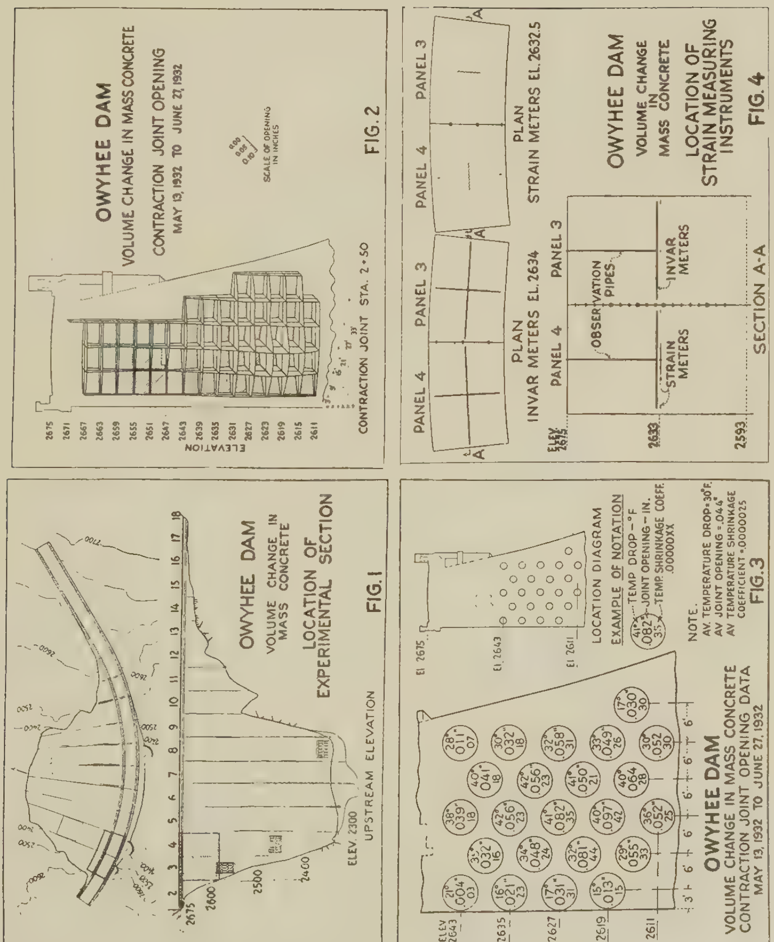
ft. long, 82 ft. high, and had an average thickness of 35 ft. (Fig. 1). Suitable devices of three different types for measuring the joint opening between panels 3 and 4, and the volume change or shrinkage within each panel were installed, 90 of these devices being mounted across the contraction joint and 70 within the mass at various points.

Fig. 2 shows an elevation of the contraction joint between panels 3 and 4, on which is imposed an isometric drawing of the contraction joint opening. The diagonal lines represent, in accordance with the scale shown at the side of the section, the amount of the contraction joint opening after river water had been pumped through the section for 40 days. Attention is directed to the shape of a horizontal section through the contraction joint—wide in the center and narrow at either side. If this joint is not grouted before the reservoir is filled, when the water load comes on the dam the arch load instead of being distributed across the entire arch section will be concentrated on either side, thereby increasing materially the unit stresses in the arch concrete.

The reason the lower part of the joint opened more than the upper is that water was pumped through the lower section longer and the temperature difference between river water and concrete was greater early in the experimental period, pumping being started May 13 before the river water had warmed up much. Pumping was continued both in the upper and lower parts of the experimental section until the rising temperature of the river water and the falling temperature of the concrete did not permit any further extraction of heat. The wider joint opening at the center, as compared with the edges, was due to the fact that more heat was available near the center for extraction by artificial cooling, whereas at the edges or corners of the blocks considerable heat was lost to the air. Panel 3 being placed first and exposed on all four sides lost more heat to the air than panel 4 which was poured last and exposed on two sides only.

Fig. 3 gives part of the numerical values used in plotting the joint opening shown in Fig. 2. The upper figure in each circle is the temperature drop of the concrete surrounding the joint measuring device. The central figure is the joint opening in inches at the location of the circle. The lower figure is the significant part of the temperature shrinkage coefficient as computed from the figures above. The average temperature drop for the lower two-thirds of the joint was 30° F., the average joint opening was .044 inches, and the temperature shrinkage coefficient, as computed from these averages, was .0000025. The difference between this value and that obtained in the laboratory for the coefficient of expansion was apparently due largely to restraint of

FIG. 1-4—VOLUME CHANGE IN MASS CONCRETE



one form or another and, in a limited measure, to the relief of residual compression in the concrete at the time cooling was started.

For the measurement of shrinkage in the mass, long invar rods were suitably mounted in embedded pipes, both radially and circumferentially, in the concrete of both panels as shown in the sectional plan views in the upper part of Fig. 4. Just below the invar rods were embedded strings of strain meters bolted together as shown in section AA, which is a vertical circumferential section taken at the cross in the invar meters.

Fig. 5 gives a comparison of the mass concrete shrinkage measurements. The upper curve is the average temperature of the concrete surrounding the instruments plotted against time. In the lower part of the figure are shown smooth dotted curves which are theoretical openings of the contraction joint according to the time temperature curve above, and are placed there solely as guide lines for the interpretation of the other curves shown. The smooth solid curve is the average shrinkage value of the 22 strain meters located under the circumferential invar meters. The solid jagged curve is the average shrinkage of the two circumferential invar meters. The dotted jagged curve is the contraction joint opening between panels 3 and 4 indicated by the contraction joint meter located across the joint between the two circumferential invar meters. Thus there is combined in this figure a comparison of three radically different types of volume change measuring devices. The gage length of the circumferential invar meters is roughly 45 ft. whereas the gage length of the strain meters is 10 in., and yet the shrinkage of the entire mass as indicated by both instruments is very close. The contraction joint opening, while not necessarily the same as the indicated shrinkage of the mass, is a very good check on the performance of the other instruments. Thus it appears that a temperature shrinkage coefficient of .000003, or less, is justified for use in estimating contraction joint openings. These five figures give the high-lights from nearly 100,000 individual observations and 150 drawings which were made in connection with this volume change experiment.

After the contraction joints in Gibson Dam had been grouted, 45 cores were diamond-drilled from across the contraction joints in various parts of the dam to determine the thickness of grout film and also its quality. The average thickness of film obtained from these cores was found to check the results obtained at Owyhee. In other words, the average temperature shrinkage coefficient obtained in the field is, by virtue of restraint and other field conditions, appreciably

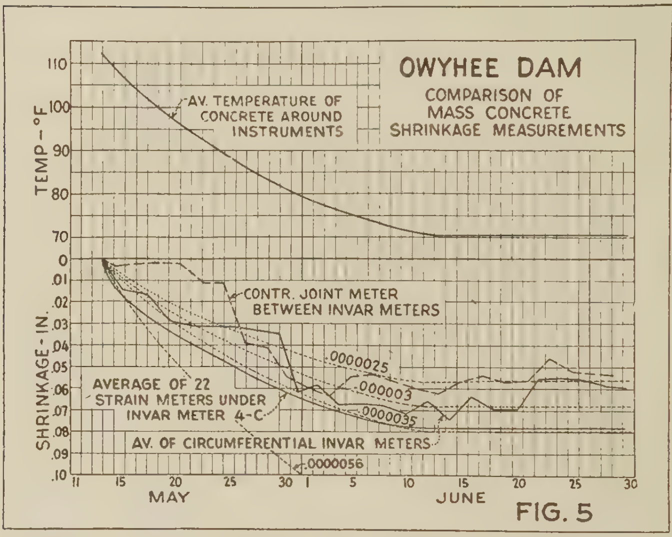
less than the coefficient of expansion (or contraction) obtained in the laboratory where the test specimen is free to move in response to temperature changes.

CONTRACTION JOINT GROUTING

For a better understanding of the flow of cement grout during the grouting of a contraction joint in a dam it was decided to conduct a series of grout experiments on a laboratory specimen in which the joint opening and all surrounding conditions could be closely controlled. In these grouting experiments, conducted in the Denver laboratories, fineness of the cement, water-cement ratio of the grout, width of contraction joint opening, grout pressure within the joint and shearing strength of the grout film have been studied. For this investigation a split cylinder of mass concrete 5 ft. in diameter by 7 ft. high, cast to simulate a contraction joint in a dam, was so constructed that the thickness of grout film could be easily and accurately controlled.

Fig. 6 shows the split cylinder, one half of which is slung from a trolley beam in such a manner that the halves of the cylinder may be spread apart to view the results of the grouting of the joint. The same pipe and fittings were used for this experimental specimen that are used in grouting the joints in a dam. However, a removable rubber grout stop was used in this experiment in place of the copper stop to prevent leakage of the grout from the joint. Metal shims spotted at various points on the joint face were used to control the thickness of the grout film.

To confine the grout within the contraction joint copper grout seals or stops, are embedded across the contraction joint near the faces of the dam. Some difficulty has been experienced in obtaining good bond between the copper and the concrete so as to prevent leakage around the grout stop. Fig. 7 shows the type of specimen used in the study of a grout stop that is more efficient and easier to install than the one which has always been used. On all dams grouted to date the bureau has used a copper stop so formed as to permit bending of the copper loop as the joint opens and closes. The grout stop shown in the figure is a straight piece of copper extending 3 in. into the concrete on each side of the joint, and is coated with an emulsified asphalt preparation. This coating is about 1/16 in. thick. When the joint opens the asphalt coating yields without breaking the bond either with the copper or the concrete. Grout pressures up to 400 p. s. i. have been introduced into these specimens, in which the joint opening was gradually increased to 1/4 in. without any leakage around the stop although leakage did


OWYHEE DAM
COMPARISON OF MASS CONCRETE SHRINKAGE MEASUREMENTS
TEMP – °F
SHRINKAGE - IN.
AV. TEMPERATURE OF CONCRETE AROUND INSTRUMENTS
CONTR. JOINT METER BETWEEN INVAR METERS
AVERAGE OF 22 STRAIN METERS UNDER INVAR METER 4-C
AV. OF CIRCUMFERENTIAL INVAR METERS
.0000025
.000003
.0000035
.0000056
MAY
JUNE
FIG. 5

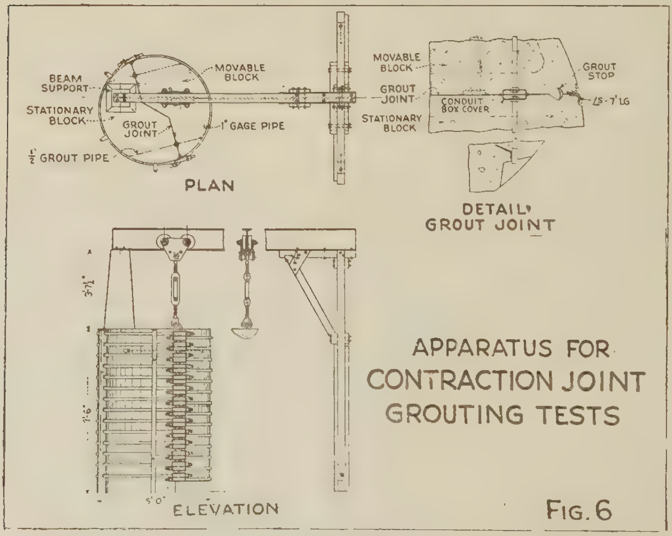

BEAM SUPPORT
STATIONARY BLOCK
GROUT JOINT
½ GROUT PIPE
MOVABLE BLOCK
1" GAGE PIPE
PLAN
MOVABLE BLOCK
GROUT JOINT
STATIONARY BLOCK
CONDUIT BOX COVER
GROUT STOP
DETAIL GROUT JOINT
ELEVATION
APPARATUS FOR CONTRACTION JOINT GROUTING TESTS
FIG. 6

Fig. 5-6

take place through the concrete along the pipe and fittings. These same specimens were used to study the water-cement ratio of the grout and to ascertain how much of the water in the grout could be forced into the adjoining concrete.

The grout studies have yielded the following pertinent facts:

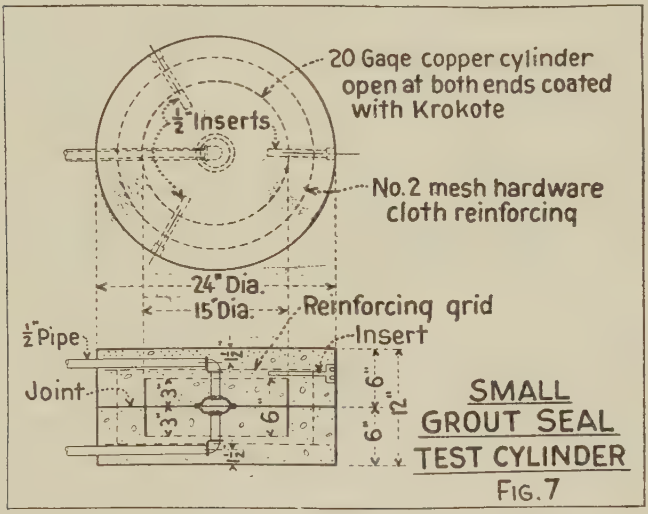


FIG. 7

(1) To obtain satisfactory flowage in grouting a contraction joint, the cement should be passed through a 200-mesh sieve immediately before the grout is mixed.

(2) For cracks of the order of .05 in. a grout in which the ratio of water to cement is as 75 to 100 by volume gives the most desirable film. A water-cement ratio of 1.00 contains too much free water that cannot be forced into the concrete or otherwise dissipated.

(3) Joint pressure of 90 per cent efficiency and joint coverage of 99 per cent are easily obtained with a grout of water-cement ratio of .75 and with a film thickness of the order of .015 in.

(4) Shear tests on grout films fog-cured at 70° F. for 28 days demonstrate that increasing the pressure within the joint immediately after it is filled does not appreciably increase the shearing value of grout films of equal initial water-cement ratio.

(5) With the apparatus used for determining punching shear, an average shearing value at 28 days of films fog-cured at 70° F. is 900 p. s. i.

PORTLAND CEMENT

The problems introduced by the cement are next in order of importance. An ideal cement would be one which generated no heat during the setting process, and consequently produced no change in volume. Since the realization of this ideal does not appear to fall within the realm of possibility, the only solution of the problem lies in reducing to a minimum the objectionable characteristics of the product.

The effect of chemical composition of portland cement on strength and certain other physical properties has received thorough consideration, both in this country and abroad, but the effect on the generation of heat during the hardening process, on volume change, permeability and durability, has until recent years received only meager attention.

After the preliminary investigation by the Bureau of Standards had been completed and the results studied, it was decided that instead of going ahead with the "comparison of cements" tests as originally planned, a more exhaustive research was desirable. This further research was to deal with the degree of control of chemical composition and fineness of grinding commercially practicable.

The cement companies in Southern California interested in supplying cement for Hoover Dam, offered their facilities for the manufacture of the laboratory cements and the grinding of both laboratory and commercial cements. Twenty of the former and 25 of the latter were prepared for test. An exhaustive investigation of these cements has been made at the University of California, in which chemical composition, heat generation, compressive strength, fineness of grinding, volume change and durability have been studied. These studies, however, have been largely confined to cement pastes, mortars and small concrete specimens.

In September, 1932, after a thorough investigation and analysis of all investigations to date, cements of chemical composition suitable for the mass concrete conditions to be encountered at Hoover Dam, and for the "comparison of cements" tests to be made in Denver, were designed, and later manufactured at the cement mills in Southern California. Heat of solution tests on cement paste and adiabatic calorimeter tests on the full mass concrete mix are now being carried on, the former at Berkeley and the latter at Denver. From the results of these tests it should be possible in the future to calculate the heat

rise in mass concrete from the heat of solution test on cement paste or possibly from the oxide composition alone.

Better to control the contraction joint openings and subsequent grouting operations, consideration is being given to the use of a low heat cement in the summer, a higher heat cement in the winter and a blend of the two for intervening periods. The construction of a cement-blending plant is being considered to blend not only high and low-heat cements, but also to blend cements of the same heat-generating qualities from different mills, so as to eliminate differences in composition, specific surface, color and temperature, and to produce a concrete mixture having essentially uniform workability, setting, hardening, strength and heat characteristics.

As soon as the tests now in progress for the "comparison of cements" have advanced far enough to yield the information desired, the specifications, of which a tentative draft was sent out November 1, 1932, will be put into final shape and invitations for bids issued.

MASS CONCRETE STRENGTH

In the design of a dam, unit strength in the concrete is one of the first questions that must be considered, since the physical dimensions of the structure are dependent upon the value of the unit adopted. The unit strength that is desired is not the strength indicated by 6 x 12 in. test specimens in which the maximum size of aggregate is 1½ in., but is the strength of representative specimens of the mass containing all sizes of aggregate up to the maximum. For this reason the bureau has for years sought to obtain data on the relation of large to small cylinders and on the effect of increasing the maximum size of aggregate to include cobbles up to 9 in. in diameter. This limiting size of cobbles is of interest because it was concluded many years ago that it was not economically practicable to introduce any larger rock into mass concrete than could be run through the mixer.

Hoover Dam offered the first opportunity to carry out a comprehensive study along this line, the cost having been prohibitive for smaller structures. When this series of mass concrete compressive tests is completed the economic effect of the introduction of cobbles will be better understood and from the analysis of the relation of large to small cylinders it will be feasible to secure in representative samples of the mass the unit strength desired and to control this unit strength by means of the usual small test cylinders which are so convenient and economical to handle. The mass concrete mix proposed for Hoover Dam and referred to as the full mass mix contains one barrel of cement per cubic yard of concrete and is composed of 1 part by weight of

cement, 2.45 parts fine aggregate, and 7.05 parts coarse aggregate, the latter including everything from No. 4 to 9 in. in diameter.

THERMAL PROPERTIES OF MASS CONCRETE

To apply to concrete the laws of the flow of heat in solids, it is necessary to know the thermal properties of the concrete. These are well known for a large variety of materials, especially those used in insulation, but practically no data were available on the thermal properties of concrete in a near saturated condition as it occurs in massive hydraulic structures.

When it was decided to give consideration to the accelerated shrinkage of the mass concrete in Hoover Dam, so that the contraction joints could be grouted during the construction period and the structure thus rendered more nearly monolithic, it became necessary to obtain the thermal properties of the concrete so that the cooling system might be economically designed. To date these properties have been determined for the concrete materials in Hoover, Owyhee, Gibson, Ariel and Bull Run Dams. The thermal property determinations made during the preliminary heat flow studies, have been very closely checked by the recent laboratory results. The variation in the values for specific heat and density are comparatively small for a wide range of mineral content in the aggregates but the variation in the corresponding values of conductivity are surprisingly large, ranging from 0.88 to 1.67* for the materials tested. Thus it will be seen that mineral content is the controlling factor in fixing conductivity values. The apparatus used for the determination of specific heat, conductivity and diffusivity of saturated mass concrete is entirely new and was developed in the Denver laboratories.

PERMEABILITY

The flow of water through concrete has been exhaustively studied by a comparatively large number of organizations. The experiments, however, have been conducted on mortars and concretes containing aggregates of small size rather than upon specimens of mass concrete containing all sizes of aggregate up to and including cobbles.

One of the principal reasons for including permeability tests in the research for Hoover Dam was to determine whether additional precautions should be taken to render the upstream face of the dam more nearly impervious than the regular mass concrete mix would normally make it.

*B. t. u. per hr. per sq. ft. per lin. ft. per degree F.

The tests have not progressed far enough to warrant definite conclusions but the early indications are that the proposed mass mix is sufficiently watertight for use in any portion of the dam.

FUTURE TESTS

It will be noted during the presentation of this series of papers that those problems of most vital importance to Hoover Dam from the construction standpoint have been attacked first. Not that such subjects as stress under axial, biaxial and triaxial loads, bond and sliding friction, shear, uplift and durability are any less important than what is being done, but they are not as tangible, and the development of testing apparatus and technique presented some real problems to solve before satisfactory results could be obtained.

As a closing thought in regard to future tests it is the author's opinion that the investigation of durability from a chemical standpoint offers one of the most important fields of research for the cement industry in achieving permanent concrete.

Readers are referred to the JOURNAL *for October (Vol. 30), for discussion which may develop. Such discussion should reach the Secretary by August 1, 1933.*

An 8-Hour Accelerated Strength Test For Field Concrete Control*

BY O. G. PATCH†

Two outstanding objects are recognized in making field test specimens of concrete on major construction jobs. One is to obtain a knowledge and record of the quality of the concrete entering the work. The second is to aid in controlling the quality of the concrete as it is being manufactured.

While the standard-cured 7 and/or 28-day specimens commonly used are desirable for checking strengths with approved standards and for comparison with other jobs, they sometimes have only a post mortem value when it comes to controlling the quality of concrete during construction.

To get the earliest possible information as to potential strength in concrete being or to be manufactured for Hoover Dam and auxiliary structures, 8-hour accelerated tests were undertaken at the Bureau of Reclamation's testing laboratory near the damsite. Several methods were tried experimentally, including low and high-pressure steam, hot water maintained at the boiling point, and the method finally adopted which requires a standard quantity of water of standard temperature in an insulated vat of standard heat-retaining capacity. This latter method not only gave the best results, but the equipment was the simplest to build and operate.

The general construction of the hot-water unit is shown by Fig. 1. The tank is filled with water to the overflow pipe, which is left open, and the heating coil is turned on. If water is cold, about 2 hours are allowed to raise the temperature to the boiling point. A sample of concrete is taken from the mixer, and after desired slump tests are taken, the cylinder specimen is made. One-half hour after taking the concrete from the mixer the curing vat overflow pipe is closed, the electric water-heater is turned off, and the concrete specimen is placed in the boiling water. The heavy insulated lid is then put on, and a record is made on the blackboard on the top of the lid showing the

*Presented at the 29th Annual Convention, Chicago, Feb. 21-23, 1933, as a part of series on Hoover Dam research.
†Bureau of Reclamation, Boulder City, Nevada.

hour placed, the hour to remove and the hour to break. An alarm clock is used for calling attention to the time for removal. After removal, at the expiration of 7 hours, the specimen is allowed to set in the 70-degree curing-room to cool for an hour before breaking. In the operation of the units the water temperature drops from about 195 degrees, after the immersed specimen warms up, to about 175 degrees at the end of the 7-hour period.

The 7-hour curing period was chosen so that a complete curing cycle would be well within an 8-hour shift, including time to reheat the water to the boiling temperature and get another specimen in place by the corresponding time of the following shift. If not operating 3 shifts, 24-hour curing periods could be adopted if preferred.

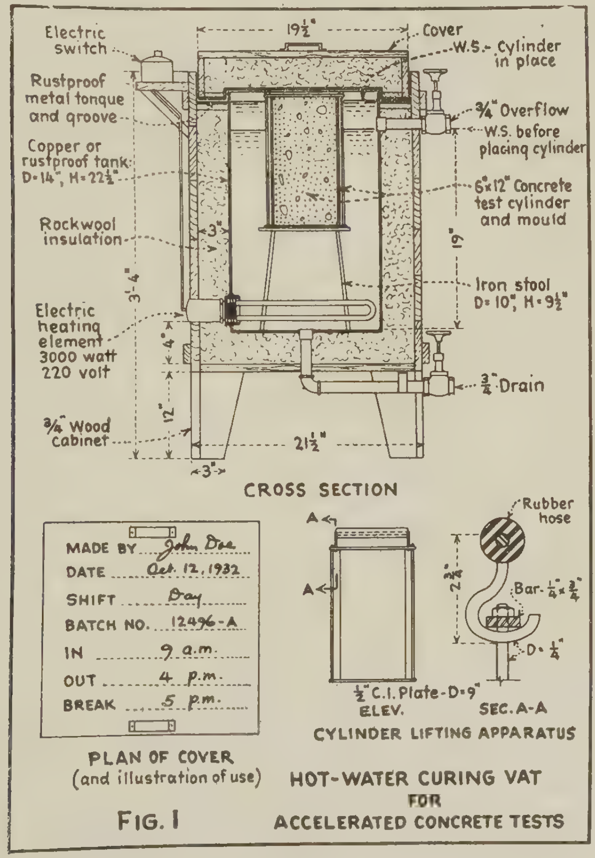


FIG. 1

TABLE No. 1

1	2	3	4	5	6	7	8	9	10
MIX USED	VARIATIONS	BRK. STRENGTH #/IN.²			% 28 DAY STR.		% OF 3" SLUMP STR.		
		HOT WATER 8 HR.	STANDARD 7 DAY	STANDARD 28 DAY	HOT WATER 8 HR.	STANDARD 7 DAY	HOT WATER 8 HR.	STANDARD 7 DAY	STANDARD 28 DAY
1½ inch max. 5 sacks per cubic yard. 1 : 2.7 : 5.3	3 inch slump	743	-------	2935	25	----	100	-----	100
	6 inch slump	584	------	2475	24	----	79	----	84
	20% less cem.	495	-----	2495	20	----	67	-----	85
	20% extra cem.	981	-----	3800	26	----	131	----	130
	#4 replac. ¾"	831	-----	2555	33	----	112	----	87
	¾" replac. #4	769	------	2720	28	----	103	-----	93
1½ inch max. 4 sacks per cubic yard. 1 : 3.3 : 6.7	3 inch slump	601	1645	2405	25	68	100	100	100
	6 inch slump	477	1290	1800	27	72	79	78	75
	20% less cem.	424	1160	1770	24	66	71	71	74
	20% extra cem.	822	2175	3110	26	70	137	132	129
	#4 replac. ¾"	522	1450	2030	26	71	87	88	84
	¾" replac. #4	672	1540	2045	30	69	112	94	93
3 inch max. 5 sacks per cubic yard. 1 : 2.3 : 5.7	3 inch slump	875	2120	3080	28	69	100	100	100
	6 inch slump	725	1840	2630	28	70	83	87	85
	20% less cem.	699	1555	2165	32	72	80	73	70
	20% extra cem.	990	2335	3155	31	74	113	110	102
	#4 replac. ¾"	849	2100	2820	30	75	97	99	92
	¾" replac. #4	875	1575	2025	43	78	100	74	66
	¾" replac. 1½"	787	------	3095	25	----	90	----	100
	1½" replac. ¾"	-----	-----	2880	----	----	----	-----	94
3 inch max. 4 sacks per cubic yard. 1 : 2.7 : 6.8	3 inch slump	637	1500	2555	25	59	100	100	100
	6 inch slump	592	1520	2430	24	63	93	101	95
	20% less cem.	531	1010	1715	31	59	83	67	67
	20% extra cem.	920	2575	2670	34	96	144	172	105
	#4 replac. ¾"	433	1800	2425	18	74	68	120	95
	¾" replac. #4	725	1130	1875	39	60	114	75	73
	¾" replac. 1½"	566	1650	2425	23	68	89	110	95
	1½" replac. ¾"	707	1495	2390	30	62	111	100	94

NOTE: Each test value is the result obtained from only one cylinder.

Before deciding on the adoption of 8-hour accelerated curing, a short series of tests was run using this method, varying the cement content, water cement ratio, and gravel gradings of the specimens. In practically all instances the mix changes were reflected very consistently in the resulting strengths. A summary of these preliminary tests is given in Table 1. The first column shows the mixes tested, 1½ in. maximum and 3 in. maximum material, of 4 sacks per yard and 5 sacks per yard cement content. The second column shows the variations made in each mix to determine whether such changes would be reflected as consistently in the 8-hour tests as in the 7-day and 28-day cylinders. The next 3 columns show actual unit compressive strengths obtained. Columns 6 and 7 show the 8-hour and 7-day strengths as percentages of the corresponding 28-day strengths, the average of all 8-hour tests being 28 per cent and all 7-day tests 70 per cent of the 28-day strengths. In general the variation from the average percentage is not essentially different with the 8-hour and 7-day tests, indicating that the 8-hour test may prove to be practically as reliable as the 7-day test in predicting 28-day results. A comparison of the last column on the chart with the two preceding columns shows how consistently, in general, the effect of mix variation in the standard 28-day cylinder strengths is predicted by the 8-hour and 7-day cyl-

inders. There are some noticeable exceptions in both 8-hour and 7-day columns, but when it is remembered that the results are from tests of only one specimen each, such discrepancies are to be expected.

These preliminary tests were limited to a range of possible mixes for diversion tunnel lining, and to such variations in the mix as might occur in actual operation at the Hoover Dam concrete plant. It would of course be of great interest to see a more comprehensive program carried out to test the various possibilities and limitations of this method.

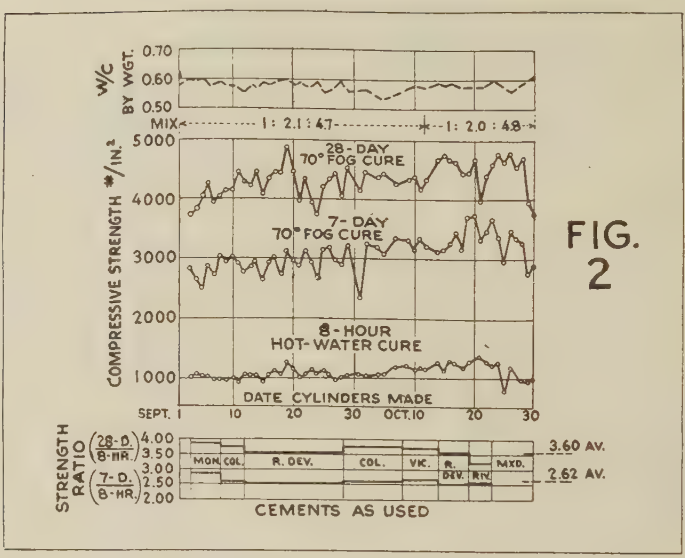
A section of the current field concrete progress sheet is shown on Fig. 2 which gives a comparison of the results with 8-hour, 7-day and 28-day specimens made daily from the same batch of concrete. It will be noted that the strength curves for the three ages show similar trends. It will also be noted that the curves cover a two-months period during which 5 different brands of cement were used. While these cements may occasionally have been mixed, owing to incomplete emptying of the bins at changes of brands, yet it was believed that a detail study of the concrete tests during the period covered by each brand would indicate possible differences in the action of the various brands with the accelerated tests. Such differences are shown by the two curves in the lower graph (See also Table 2) in terms of strength ratios. Thus the upper curve shows that the 28-day strength is from 3.2 to 3.9 times the 8-hour strength, depending on the cement used. Similarly the lower curve shows the 7-day to 8-hour strength ratio to range from about 2.5 to 2.8

TABLE 2

Average Ratios of Strengths	Brands of Cement					Average Ratios for all Cements
	Victorville	Colton	Monolith	Red Devil	Riverside	
28-Day / 8-Hour	3.68	3.73	3.87	3.53	3.20	3.60
28-Day / 7-Day	1.41	1.46	1.36	1.45	1.26	1.39
7-Day / 8-Hour	2.64	2.59	2.83	2.51	2.54	2.62

A more comprehensive analysis of the cement factor in affecting strength ratios is contained in Fig. 3. Here the strength data for each brand of cement are plotted against water-cement ratio.

If the tests covered a greater range of water ratios, the plotted points would tend to fall along curved lines. For the comparatively short range covered, however, straight lines gave simpler relationships and were thought to be accurate enough for practical purposes. For


W/C BY WGT.
0.70
0.60
0.50
MIX 1 : 2.1 : 4.7 1 : 2.0 : 4.8
COMPRESSIVE STRENGTH #/IN.²
5000
4000
3000
2000
1000
28-DAY 70° FOG CURE
7-DAY 70° FOG CURE
8-HOUR HOT-WATER CURE
DATE CYLINDERS MADE
SEPT. 1 10 20 30 OCT. 10 20 30
STRENGTH RATIO
(28-D./8-HR.)
(7-D./8-HR.)
4.00
3.50
3.00
2.50
2.00
MON. COL. R. DEV. COL. VIC. R. DEV. RIV. MXD.
3.60 AV.
2.62 AV.
CEMENTS AS USED
FIG. 2

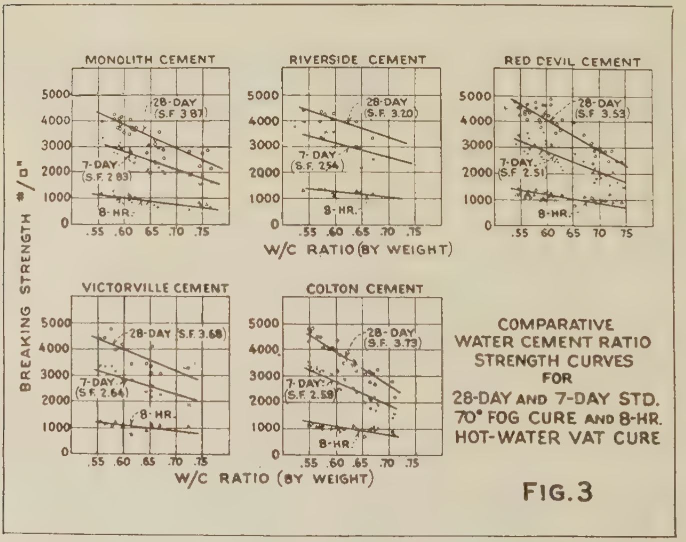

BREAKING STRENGTH #/□"
MONOLITH CEMENT
28-DAY (S.F. 3.87)
7-DAY (S.F. 2.83)
8-HR.
RIVERSIDE CEMENT
28-DAY (S.F. 3.20)
7-DAY (S.F. 2.54)
8-HR.
RED DEVIL CEMENT
28-DAY (S.F. 3.53)
7-DAY (S.F. 2.51)
8-HR.
VICTORVILLE CEMENT
28-DAY (S.F. 3.68)
7-DAY (S.F. 2.64)
8-HR.
COLTON CEMENT
28-DAY (S.F. 3.73)
7-DAY (S.F. 2.59)
8-HR.
0 1000 2000 3000 4000 5000
.55 .60 .65 .70 .75
W/C RATIO (BY WEIGHT)
COMPARATIVE WATER CEMENT RATIO STRENGTH CURVES FOR 28-DAY AND 7-DAY STD. 70° FOG CURE AND 8-HR. HOT-WATER VAT CURE
FIG. 3

FIG. 2-3

the same reasons, the slopes of the 8-hour and 7-day lines were made to conform to constant average strength ratios, even though in some cases slightly different slopes would fit the points better.

For many purposes it is probable that the grand average 28-day/8-hour factor of 3.6 will be sufficiently accurate for any of these brands. A complete research program carried out with this method may even demonstrate that an 8-hour test made with such a standard hot water unit, with a minimum of equipment and attention, will be fully as dependable and as truly indicative of the inherent quality of the concrete, as is a 28-day test made under the more or less adverse conditions of moisture and temperature control so often found, particularly at the start of a large per cent of the smaller construction jobs.

When the size of a job will warrant it, and standard curing equipment is available, a check on the 8-hour factor can be secured within 28-days, and any modification or adjustment may be made for the particular job conditions.

While this method of checking concrete control is still in its infancy, the results so far indicate a prophetic accuracy practically as great as that obtained from the 7-day tests. Needless to state, the value of 8-hour checks in the control of the manufacture of concrete is much greater than that of either 7-day or 28-day tests of equal accuracy could be. Some of the possible advantages of using the 8-hour accelerated test specimens in preference to 7-day and/or 28-day standard cured specimens for construction work are:

(a) The results are obtained almost immediately and mix corrections can be made with the minimum of lost time.

(b) Possible changes in the mix design, or alternative mixes may be tried out with very little delay.

(c) For a specified number of tests per shift only one-third the number of cylinder molds is needed, if 3 shifts are worked each day.

(d) The required curing facilities are very modest as compared to the requirements of an up-to-date temperature and humidity-controlled curing room, as only an insulated rust-proof can for each cylinder desired each shift, is necessary; also, specimens do not accumulate as in later-age tests.

(e) The hot-water units are readily portable, and can be set up any place where there are reasonably even temperature conditions during the curing period. Boiling water can be obtained by an electric heater, from a steam boiler, or heated on a stove.

(f) The variations caused by extremes of heat or cold to which the usual field specimens are often subjected during the first 24 hours are practically eliminated.

While much remains to be done in the way of research and standardization, before accelerated tests can be relied on fully for design or control purposes, it is believed that they can already be safely used to speed up tests on trial mixes, and to serve as a valuable safety check against gross irregularities in concrete production, thus saving time and relieving uncertainty. Such aid is especially appreciated at the beginning of concrete operations.

Readers are referred to the JOURNAL *for October (Vol. 30), for discussion which may develop. Such discussion should reach the Secretary by August 1, 1933.*

Relation Between Quality and Economy of Concrete*

BY INGE LYSE†

SYNOPSIS

This paper presents a summary of the inter-relation between the strength, permeability, durability, fire resistance and volume changes of concrete. It also submits a study of the relation between the strength of the concrete and the economy of plain and reinforced concrete members. It explains how a rational economical study is made possible by the constant water content theory, and how such factors as prices of cement and aggregate, strength, quality of cement and gradation of aggregates affect the economy of concrete mixes. A study has also been made of the economy of reinforced concrete members subjected to compression and flexure, and results are presented showing the inter-relation of strength of the concrete, yield-point strength of the reinforcement and the economy of the member.

INTRODUCTION

The relation between quality and economy of plain and reinforced concrete members has long been in doubt. It has been pointed out by different authors that a rich concrete mix will usually result in a more economical reinforced concrete structure than will a lean mix. As early as 1907 A. N. Talbot (1)‡ pointed out that the richer concrete mix was more economical than a leaner one for columns of a given strength. Peter Gillespie (2) and M. O. Withey (3) have also stated that the richer concrete is the more economical for reinforced concrete columns and that increased richness added strength more economically than increased longitudinal reinforcement. Arthur R. Lord (4) recently presented data showing a considerable saving when the strength of the concrete was 3000 instead of 2000 p. s. i. H. Olsen (5) concluded that the higher working stress (which requires stronger concrete and higher yield-point strength of the reinforcement) resulted in a lower cost of the structure.

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*Presented at the 29th Annual Convention, Chicago, February 21-24, 1933.
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‡References are to a bibliography at the end of the paper.

No rational study, however, has previously been given to the direct relation between the strength of the concrete and the economy of the concrete member. This study has been made possible by the establishment of the constant water requirement for concrete of a given consistency regardless of the richness of the mix (6). The unit cost of concrete may therefore be expressed in terms of the cement content, and since the strength is a function of the cement content; the unit cost is definitely related to the strength of the concrete.

NOTATION

The notation adopted for use in this paper is as follows:

A = coefficient in strength equation.
A_c = area of concrete (gross area).
A_s = Area of longitudinal reinforcement.
A_s' = equivalent area of spiral reinforcement.
a = aggregate content.
a' = change in aggregate content.
B = coefficient in strength equation.
c = cement content.
c' = change in cement content.
D = density of concrete, or portion of solids.
d = effective depth of beam.
E = cost per unit of strength.
F = total load on structural member.
f_c' = cylinder strength of concrete.
f_s = yield-point stress of longitudinal reinforcement.
f_s' = yield-point stress of spiral reinforcement.
g_a = specific gravity of aggregates.
g_c = specific gravity of cement.
g_p = specific gravity of cement paste.
g_w = specific gravity of water.
j = ratio in beam equation.
K = coefficient in strength equation.
k = ratio in beam equation.
k' = effectiveness ratio of spiral reinforcement, that is, the ratio between the strength added by the spiral and that added by the same amount of longitudinal reinforcement.
M = bending moment at maximum load.
n = ratio of modulus of elasticity of steel to that of concrete.
P = cost per unit volume.
p = percentage of reinforcement.
p_a = price of aggregates.
p_c = price of cement.
p'_c = price of concrete.
p_s = price of reinforcement.
S = strength.
s = factor of safety.
V = volume of concrete.

V_a = absolute volume of aggregates.
V_c = absolute volume of cement.
V_g = absolute volume of coarse aggregate.
V_s = absolute volume of sand.
V_w = volume of water.
v = voids in concrete, air voids plus water.
w = water content.
y = the product kj.
Z = thickness of protective cover.

HISTORICAL DEVELOPMENT

The first rational study of the factors affecting the strength of cement mortars was made by R. Feret (7). He found that the strength of the mortars was determined by the amount of cement per unit of voids (air voids + water) in the mortar and also by the amount of cement per unit of cement plus voids. Feret's experimental evaluation of the relationship was:

$$S = K\left(\frac{c}{1-V_s}\right)^2 \qquad (1)$$

where K depends upon the materials and the conditions of the test.

Assuming that the same relationship would hold for concrete, the equation becomes:

$$S = K\left(\frac{c}{1-V_s-V_g}\right)^2 = K\left(\frac{c}{1-V_a}\right)^2 \qquad (2)$$

M. O. Withey (8) in 1914 published results showing a straight line relationship between the strength and the cement-void ratio of concrete. Professor Withey's relation may be expressed by the equation:

$$S = A + B.\frac{c}{v} \qquad (3)$$

In 1918 Duff A. Abrams (9) published his water-cement ratio relationship and presented the following equation for plastic and workable concrete mixes:

$$S = \frac{A}{B^{w/c}} \qquad (4)$$

For average laboratory conditions Professor Abrams evaluated the constants in the equation:

$$S = \frac{14{,}000}{7^{w/c}} \qquad (5)$$

Since the first publication of the water-cement ratio many investigators have contributed a vast amount of information on the relationship between the strength and the water-cement ratio of the mix. The outstanding contributors are F. R. McMillan (10) and Otto Graf (11). Professor Graf expanded Abrams' water-cement ratio relationship by the use of a factor which represented the strength quality of the cement used.

At the University of Illinois experimentation on the fundamental relationships for the strength of concrete had been carried out for many years under the able leadership of Professor Talbot. In 1923 the results of these studies were published (12) by Professors Talbot and Richart. The relationship for strength was expressed as a function of the cement-space ratio or the cement-void ratio. The equation given for average conditions was:

$$S = \frac{32{,}000}{\left(1 + \frac{v}{c}\right)^{2.5}} = 32{,}000\left(\frac{c}{c + v}\right)^{2.5} \qquad (6)$$

It is noted that the Illinois results check closely Feret's results for mortar. However, the cement-void and the cement-space relations never received the attention which was stimulated by Abrams' water-cement ratio.

A study of the relationships for the strength of concrete presented above, reveals that in all cases the cement and the voids or the water, played an important role in the equations. There is therefore no fundamental difference between the several relationships obtained. Since for plastic and workable mixes the air voids in properly placed concrete are generally less than one per cent, the difference between the voids and the water content becomes so small that the void-cement ratio is very nearly equal to the water-cement ratio. W. A. Slater (13) has shown how well the Talbot-Richart void-cement ratio curve agrees with Abrams' water-cement ratio curve when the voids are equal to the water content.

The water-cement ratio relationship is now generally recognized as the criterion for the strength of concrete.

In 1930 R. L. Bertin (14) suggested the use of the specific gravity of the cement paste as a measure of the strength of the concrete. He found that the Abrams water-cement ratio curve became very nearly a straight line when the strength was plotted against the specific gravity of the cement paste. Bertin's equation for the strength of concrete is:

$$S = A.g_p - B \quad \text{(7)}$$

In 1931 it was shown (15) that when the strength of the concrete was plotted against the reciprocal of the water-cement ratio, that is the cement-water ratio, both the Abrams and the Talbot-Richart curves for plastic and workable mixes gave an approximately straight line relation. The relationship between the strength and the cement-water ratio of concrete has since been studied more fully and it has been shown that within the range of practical concrete mixes the straight line relation serves very well. It has also been shown that when the water content in a unit of concrete remains constant and the cement and the aggregate contents are the only variables, the strength increases in direct proportion to the increase in the cement content, or in other words, in direct proportion to an increase in the cement-water ratio. The relationship between strength and the cement-water ratio of concrete is therefore given by the formula:

$$S = A + B.\frac{c}{w} \quad \text{(8)}$$

This equation holds only within a range which covers all practical concrete mixes. For extremely lean as well as extremely rich mixes, the straight line relationship does not apply. This leads to the conclusion that for practical mixes the strength of the concrete is determined by the concentration of cement particles in a unit of water, which may be expressed as follows: Above a minimum number of cement particles necessary to give workability and binding strength to concrete, the strength of the concrete increases in direct proportion to the increase in number of cement particles per unit of water. Several French technical papers have also pointed out this straight line relation between the cement-water ratio of the paste and the strength of the concrete (16).

Furthermore, it has been shown (6) that for a given type and gradation of aggregates the consistency of the concrete remains nearly constant as long as the water content per unit of concrete remains the same. Thus for concrete of equal consistency the strength equation becomes:

$$S = A + B/w.c = A + K.c \quad \text{(9)}$$

The strength of the concrete is here a function of the cement content in such a way that the strength increases in direct proportion to the increase in the cement content. Equation (9) is very convenient for the design and control of concrete mixes (17) and also gives the foundation for the rational study of the economy of concrete structures.

RELATION BETWEEN STRENGTH AND OTHER QUALITIES OF CONCRETE

It has been shown above that the compressive strength of concrete increases directly in proportion to the increase in the cement-water ratio. Gonnerman and Shuman (18) have shown that the factors which determine the compressive strength also ascertain the tensile and flexural strength. They found that the tensile and flexural strengths of concrete were determined by the water-cement ratio of the paste in much the same manner as the compressive strength. It may therefore be concluded that the strength properties of concrete are primarily determined by the cement-water ratio.

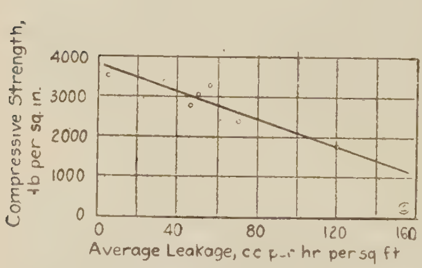

FIG. 1—STRENGTH AND PERMEABILITY OF CONCRETE MADE FROM DIFFERENT CEMENTS

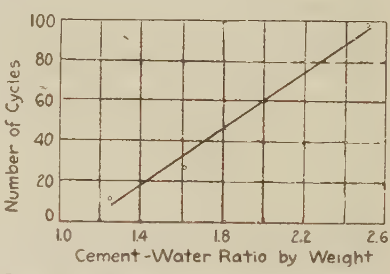

FIG. 2—RELATION BETWEEN INITIAL DISINTEGRATION AND CEMENT-WATER RATIO

The permeability of concrete has also been primarily dependent upon the same factors as those determining the strength of the concrete (19). Fig. 1 shows the relation between the strength and permeability of concretes, each of which contained a different brand of cement. It is noted that the permeability decreases in direct proportion to the increase in strength of the concrete. Mr. McMillan (20) has also shown that for a given brand of cement and a given curing of the concrete, the permeability decreases with the increase in strength.

C. H. Scholer (21) has demonstrated that the durability of concrete is to a large extent determined by the strength of the concrete. The relation between the number of cycles of freezing and thawing which will produce an initial disintegration (22) and the cement-water ratio of the concrete is shown in Fig. 2. The number of cycles increases directly with the increase in the cement-water ratio. This means that the resistance to initial disintegration increases in direct proportion to the increase in the concentration of cement in the cement paste used. The same law, therefore, which applies to the strength of concrete also applies to the resistance of initial disintegration.

The fire resistance of concrete has lately been studied extensively (23) and the outstanding results of these studies are presented in Fig. 3, where it is noted that the fire resistance of the concrete increases in direct proportion to the increase in cement content, both for ordinary aggregates and for the light weight aggregate, Haydite. It is also seen that the concrete containing light weight aggregate shows a con-

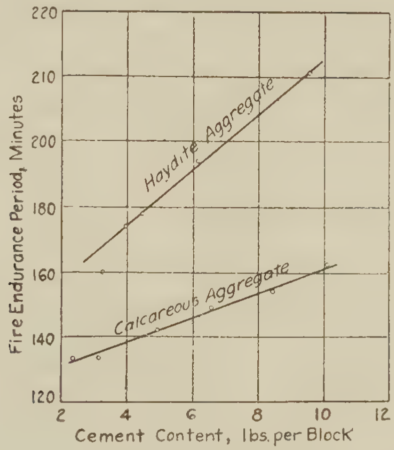

FIG. 3—EFFECT OF CEMENT CONTENT ON FIRE ENDURANCE OF WALLS OF CONCRETE MASONRY UNITS

siderably higher fire resistance for the same cement content than does concrete containing ordinary aggregates, and that the fire resistance increases more rapidly with the increase in cement content for concrete containing light weight aggregates than for concrete containing ordinary aggregates.

From these results it may be concluded that the factors which give high strength will also give high impermeability, greater resistance to freezing and thawing, and high fire resistance.

On the other hand, the volume changes due to soaking and drying of the concrete will increase with the increase in cement content (13). Therefore, the greater the richness of the concrete mix, the greater is the volume change. The properties of the concrete which produce high strength will therefore not necessarily produce low volume changes.

THE ECONOMY OF PLAIN CONCRETE

The constant water content theory enables us to make a rational study of the relationship between the economy of different concrete

mixes. For mixes having a constant water content, the amount of solids in the concrete remains the same regardless of richness of mix so that the change in cement content is accompanied by a similar change in the aggregate content. Thus the following relation is found:

$$\frac{c'}{g_c} = \frac{a'}{g_a} \text{ or } \frac{a'}{c'} = \frac{g_a}{g_c} \quad \text{.......} (10)$$

Setting up the equation for the volume of the concrete:

$$V = V_c + V_a + V_w = \frac{c}{g_c} + \frac{a}{g_a} + V_w = D + V_w \quad \text{.......} (11)$$

$$\text{since } \frac{c}{g_c} + \frac{a}{g_a} = D, \text{ from which: } a = D.g_a - c.\frac{g_a}{g_c}$$

The price of the concrete materials is:

$$P = p_c c + p_a a \quad \text{.......} (12)$$

or substituting for a:

$$P = p_c c + p_a \left(Dg_a - c.\frac{g_a}{g_c}\right) = p_a D.g_a + c.\left(p_c - \frac{g_a}{g_c} p_a\right) \quad \text{.......} (13)$$

Thus the cost of the concrete materials is a direct function of the cement content for given prices on aggregates and cement. By means of equation (13) the cost of any mix may be determined when the prices of the materials, the density of the mix and the specific gravities of the aggregates and the cement are known. Since for ordinary concrete materials g_a = about 2.65 and g_c = about 3.10, the equation is reduced to:

$$P = 2.65D.p_a + (p_c - 0.85p_a)\, c \quad \text{.......} (14)$$

If one cubic yard be used as unit of concrete, the equation becomes when p_c and p_a are given in cents per pound:

$$P = 4460D.p_a + (p_c - 0.85p_a)\, c \quad \text{.......} (15)$$

where c is given in pounds per cubic yard of concrete. Thus for given prices of the material and for a given consistency of the concrete, the cost of the materials per cubic yard of concrete is directly related to the variation in the cement content.

It has previously been shown that the strength for concrete of constant water content is given by the formula:

$$S = A + Kc \text{ or } c = \frac{S\text{-}A}{K} \quad \text{.......} (16)$$

Both the cost of the materials and strength of the concrete are therefore given as a direct function of the cement content. The cost expressed in terms of the strength becomes when p_a and p_c are given in cents per pound:

$$P = 4460D.p_a + (p_c - 0.85p_a)\frac{S\text{-}A}{K} \quad \text{.......... (17)}$$

For plain concrete members loaded directly in compression the load-carrying capacity is directly proportional to the strength of the concrete. The size of the concrete member for a given load is therefore determined by the strength of the concrete. The cross section area of the member is given by:

$$A_c = \frac{F}{s.f_c'} \quad \text{.......... (18)}$$

For a unit length of the member the volume of concrete is proportional to the cross-sectional area. Thus the volume of concrete required to carry a given load is inversely proportional to the strength of the concrete. The price of concrete per unit of strength is therefore a measure of the economy of the mix. The price per unit of strength is given by the equation:

$$E = \frac{P}{S} = \frac{1}{S}\left(4460\text{D}.p_c + (p_c - 0.85p_a)\frac{S\text{-}A}{K}\right) \quad \text{.......... (19)}$$

Fig. 4 shows the relation between the cost per cubic yard of concrete per 1000 p. s. i. of strength and the strength of the concrete for given materials and conditions of test and for five different prices of the cement. The decrease in cost for a given load with the increase in strength of the concrete is very great. A change in the cement content, which will produce a corresponding change in the strength, will easily offset small differences in the price of the cement. This figure is a strong evidence for increased economy with increased strength of the concrete It should furthermore be kept in mind that the concrete of high strength has other advantages as compared with concrete of low strength, namely, higher resistance to leakage, to fire and to the action of freezing and thawing. The only disadvantage incorporated in rich concrete over lean mixes is the larger volume changes.

In Fig. 5 the only variable introduced was that of the price of the aggregates. It is noted that a change in the price of the aggregates has a greater effect upon the economy of the mix than has a corresponding change in price of the cement. This is due to the relatively large quantities of aggregates in a unit of concrete. It is noted also that the

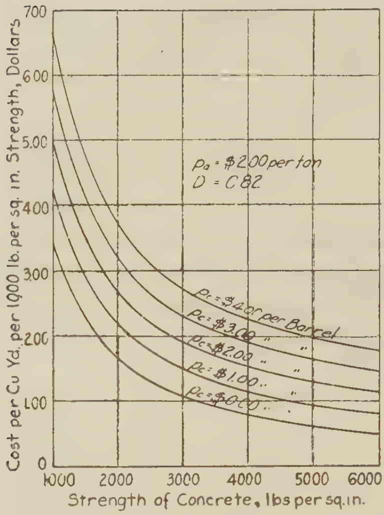

FIG. 4—RELATION BETWEEN STRENGTH AND ECONOMY OF CONCRETE FOR DIFFERENT PRICES OF THE CEMENT

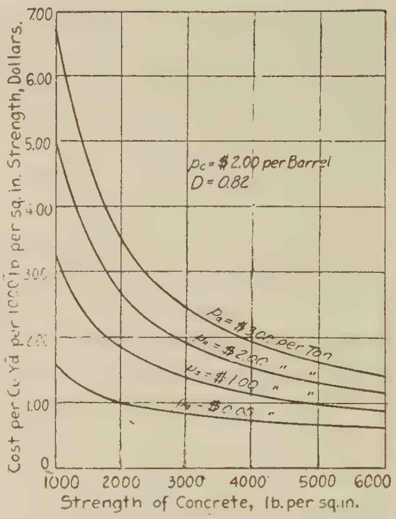

FIG. 5—RELATION BETWEEN STRENGTH AND ECONOMY OF CONCRETE FOR DIFFERENT PRICES OF THE AGGREGATES

effect of the price of the aggregates on the economy of the mix is more prominent for lean than for rich mixes. The economy of the concrete increased very much with the increase in strength of the concrete regardless of the price of the aggregates. The strength relation for the concrete used in Fig. 4 and 5 was:

$$S = -2570 + 3600.c/w$$

To study the effect of the strength-giving qualities of the materials on the cost of the concrete, three different strength relations were considered. In Fig. 6 the strengths have been plotted against the cement-water ratio for concretes containing three different cements. The strengths for a given cement-water ratio were greatly different for the different cements. In Fig. 7 the cost per cubic yard of concrete per 1000 p. s. i. strength has been plotted against the strength of the concrete for these three strength relations. The cost is seen to be slightly higher for the cement giving the lower strength, but the difference is not marked. A slight increase in the richness of the concrete would easily offset the small differences in cost due to the strength qualities of the cements.

In Fig. 8 the variation in cost with strength of concrete is shown for different densities of the concrete. In preparing this figure it was

assumed that the strength of the concrete increased in direct proportion to the increase in the cement-water ratio of the paste, regardless of the gradation of the aggregates and the consequent variation in the density of the concrete. The density of the concrete was varied from as high as 0.85 to as low as 0.75. It is noted that these variations in the density produced only slight differences in the cost of the concrete.

The outstanding result of this study of plain concrete is the great increase in economy with the increase in the strength of the concrete.

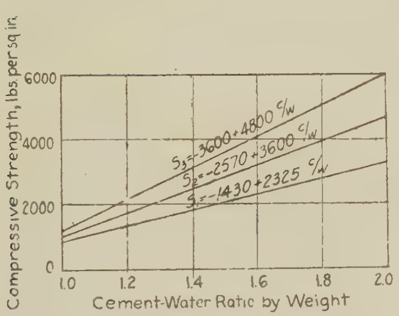

Fig. 6—Strength curves for 3 different cements

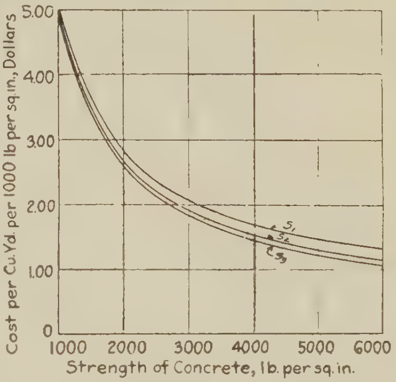

Fig. 7—Effect of strength quality of cement on economy of concrete

In the practical application of the above study the cost of mixing, placing, curing and possible finishing of the concrete and also the cost of formwork and labor contribute to the total cost of the final structure. However, the cost of preparing, handling and curing the concrete is nearly the same for rich and for lean mixes. The cost of formwork, labor, and finish may be somewhat different for the different concretes, since the cross-sectional area of the member changes with the strength of the concrete and is greater for lean than for rich mixes. However, in most cases the differences in the cost of these items are so small that for studies of the nature presented in this paper they may be neglected.

ECONOMY OF REINFORCED CONCRETE COLUMNS

In a reinforced concrete member the cost per linear unit is made up of the cost of the concrete, the cost of the reinforcement, the cost of forms and the cost of possible finish. Where the total area of the member is in compression, both the concrete area and the steel area will contribute their full load-carrying capacity. The strength of such a

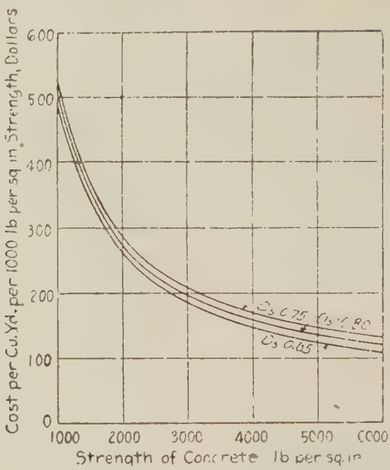

FIG. 8—EFFECT OF GRADATION OF AGGREGATES ON ECONOMY OF CONCRETE

member is made up of the individual strengths of the concrete and the reinforcement. The effectiveness of the concrete in a reinforced concrete column has been found to be approximately 85 per cent (24) of the cylinder strength. Thus for a tied reinforced concrete column the ultimate strength is given by:

$$S = 0.85f'_c A_c + f_s A_s \qquad (20)$$

For a spirally reinforced concrete column the ultimate strength is given by:

$$S = 0.85f'_c A_c + f_s A_s + k' f'_s A'_s \qquad (21)$$

If the load carried by the spiral is equal to, or less than the load carried by the protective concrete shell outside the spiral, the spiral will not add to the strength of the column. The effectiveness* ratio, k', becomes zero and the formula for the spirally reinforced columns is equal to the formula for the tied column. In the following study the strength contributed by the spiral reinforcement has been assumed to be equal to or less than the strength of the concrete shell.

In making an economical study of a reinforced concrete column, the following cost equation is used:

$$P = p'_c A_c + p_s A_s + \text{Forms} + \text{Finish} \qquad (22)$$

where P is final cost per unit of length of column. Since the cost of

*The effectiveness is here measured in terms of the strength added by the spiral in excess of the strength added by the protective shell.

forms and finish does not vary appreciably with the size of the columns used, it has been neglected in the following study. The first term on the right-hand side of the equation is studied under plain concrete except that the cost is 17½ per cent greater than for plain concrete, due to the fact that only 0.85 of the cylinder strength is available in the reinforced column.

The most economical column is the one having the lowest cost per unit of strength. The unit cost of concrete is:

$$\frac{p'_c \cdot A_c}{0.85 f'_c \cdot A_c} \ldots\ldots$$

and the unit cost of steel is:

$$\frac{p_s \cdot A_s}{f_s \cdot A_s},$$

in which the steel area A_s in the numerator and denominator is equal for columns having welded reinforcement, but in which the steel area in the numerator is about 1.20 times the area in the denominator for columns having spliced reinforcement. Thus for columns having welded reinforcement we have

$$\frac{p'_c}{0.85 f'_c} \text{ and } \frac{p_s}{f_s} \ldots\ldots$$

are unit cost of concrete and steel respectively, per unit of length. In order to have a lower unit cost for reinforcement than for concrete $\frac{p_s}{f_s}$ must be less than $\frac{p_c'}{0.85 f_c'}$ Since p_s is usually given in terms of tons and p_c' in terms of cubic yards, it is necessary to relate these terms.

1 cu. yd. steel = 490 lb. x 27 = 13,200 lb. = 6.6 tons. Thus $\frac{p_s}{f_s}$ must be less than $\frac{1}{6.6} \cdot \frac{p'_c}{0.85 f'_c} = \frac{p'_c}{5.6 f'_c}$, or the price p_s must be less than $\frac{1}{5.6} \cdot \frac{f_s}{f'_c} \cdot p'_c =$ $0.18 \frac{f_s}{f'_c} p'_c$. For $f_s = 45{,}000$ and $f'_c = 2000$, p_s must be less than $0.18 \frac{45000}{2000} p'_c =$ $4.05 p'_c$. For $f_s = 80{,}000$ and $f'_c = 2000$, p_s must be less than $0.18 \frac{80000}{2000} p'_c =$ $7.20 p'_c$.

In Fig. 9 the values of the cost ratio are given for several yield-point strengths of the reinforcement and for several strengths of the concrete.

For columns having spliced reinforcement the cost ratio becomes, when the amount of splicing is 20 per cent of the theoretical amount of reinforcement:

$$p_s = \frac{1}{1.20} \cdot 0.18 \frac{f_s}{f'_c} \cdot p'_c = 0.15 \frac{f_s}{f'_c} \cdot p'_c \ldots\ldots\ldots\ldots\ldots$$

Under plain concrete it was pointed out that the most economical concrete mix was that which produced the greatest strength. It is noted from Fig. 9 that the greater the yield-point of the reinforcement, and the lower the strength of the concrete, the most chance there is that the reinforcement may be more economical than the concrete for carrying a definite load.

By means of this relationship between the cost of steel and concrete it is a simple matter to compute the most economical reinforced concrete column.

If a reinforced concrete column contains spiral reinforcement which will give strength in excess of the strength of the protective shell, a study of the economy of the spiral can be made in the same manner as shown above for the longitudinal reinforcement. However, the economy of the spiral reinforcement may be considered more directly. If the cost of the spiral reinforcement be less than the cost of the longitudinal reinforcement per unit of load carried, the cost ratio between the spiral and the longitudinal reinforcement must be less than the effectiveness ratio of the spiral. Thus, if the effectiveness ratio of the spiral is 1.0, the cost of the spiral reinforcement in place must be less than the cost of the equivalent percentage of longitudinal reinforcement if a more economical column is desired.

ECONOMY OF REINFORCED CONCRETE BEAMS

It is obvious that in reinforced concrete slabs and rectangular beams

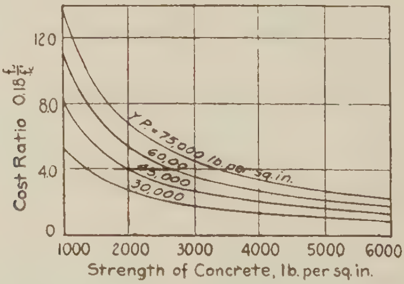

FIG. 9—RELATION BETWEEN STRENGTH OF CONCRETE AND ECONOMIC COST RATIO FOR REINFORCED CONCRETE COLUMNS

the most economical design results when both the steel and the concrete are fully utilized, that is, when the maximum stress in the steel corresponds to its yield-point stress and the maximum stress in the concrete corresponds to its ultimate strength, provided the bending moment governs the size. If both materials were not fully utilized the steel would reach its yield-point stress before the concrete reached its ultimate strength, or vice versa, and the depth or the amount of reinforcement would be greater than that necessary for its load-carrying capacity. The depth of the section or the amount of reinforcement could be adjusted until the steel and concrete stresses were balanced without affecting the strength of the section.

It has been shown (25) that the compressive strength of the concrete in flexure is considerably greater than that in direct compression when straight line stress distribution is used in the computation. The maximum compressive stress in reinforced beams corresponded to approximately 1.5 times the cylinder strength for concrete of strengths of more than 2000 p. s. i. and this value has been used in the following study. It has also been shown (25) that the location of the neutral axis for beams of different strengths of concrete does not vary greatly. This is due to the fact that the location of the neutral axis is not only determined by the ratio of the moduli of elasticity of steel and of concrete, but also by the percentage of reinforcement. The position of the neutral axis is given by

$$k = \sqrt{2pn + (pn)^2} - pn \ldots\ldots$$

Since an increase in the strength of the concrete will decrease the ratio n but require an increase in the percentage p, the position of the neutral axis is only slightly affected. The location of the neutral axis has therefore been taken as fixed in the following study. Per unit width of slab the ordinary straight line stress distribution gives:

$$1.5f'_c = \frac{2M}{k.j.d^2} = \frac{2M}{y.d^2} \ldots\ldots (23)$$

$$\text{or } d^2 = \frac{2M}{1.5.yf'_c} = \frac{4}{3}\frac{M}{y.f'_c}$$

$$d = 2\sqrt{\frac{M}{3.y.f'_c}} \ldots\ldots (24)$$

The effective depth of the beam thus decreases inversely with the square root of the cylinder strength. Since the cost equation per unit length of a reinforced concrete beam of unit width is:

$$P = d.p'_c + A_s p_s + Z.p'_c + \text{Forms} + \text{Finish} \ldots\ldots\ldots\ldots (25)$$

the cost of the slab for a given condition decreases with the decrease in the effective depth. If the cost of forms and finish be considered equal for different depths, the variation in the strength of the concrete affects only the first and third terms of the equation. Neglecting for the present, the cost of the protective cover, $Z.p_c'$, a study will be given to the first term:

$$dp'_c = p'_c.2.\sqrt{\frac{M}{3.y.f'_c}} = \frac{2p'_c.\sqrt{\frac{M}{3.y}}}{\sqrt{f'_c}} \ldots\ldots\ldots\ldots$$

For a given moment, M, the cost of the concrete in the effective depth of the section is seen to vary inversely with the square root of the strength of the concrete. The cost of the protective cover, $Z.p_c'$, will increase with the strength of the concrete and the amount of steel will have to be increased to prevent failure in the reinforcement. The amount of reinforcing steel is given by:

$$A_s = \frac{M}{j.d.f_s} = \frac{M}{2.j.\,f\sqrt{\frac{M}{3.y.f'_c}}} = \frac{\sqrt{3.M.y.f'_c}}{2j.f_s} \ldots\ldots\ldots\ldots (26)$$

The amount of reinforcement will thus vary directly with the square root of the strength of the concrete. Since the relative increase in the amount of steel is equal to the relative decrease in the volume of effective concrete, the total cost of the section may either increase or decrease with the increase in strength of the concrete, depending upon the relative costs and amounts of concrete and steel. Generally the decrease in cost of the concrete will more than offset the increase in the cost of the steel and of the protective cover. To illustrate the relation between the variation in the cost of the section and the strengths of the concrete for given conditions Fig. 10 has been prepared. In this figure relative costs of the section for different strengths of the concrete are shown for the following conditions.

$M_{ult.}$ = 100,000 lb.-in. per inch of width

$f_s = 40{,}000$ p. s. i.

$j = 0.82$

$y = k.j = 0.44$

$D = 0.82$

$$p'_c = 4460D.p_a + (p_c - 0.85p_a)\frac{S\text{-}A}{K} = 3660p_a + (p_c - 0.85p_a)\frac{S + 2570}{12}$$

in which:

p_a = 0.10 cents per pound, and p_c = 0.50 cents per pound

Then:

$$p'_c = 366 + 0.415\frac{S + 2570}{12} = 366 + 89 + 0.0346S = 455 + 0.0346.S, \text{ cents}$$

per cubic yard.

$p_s = 3\frac{1}{2}$ cents per pound, and Z = 1.0 in.

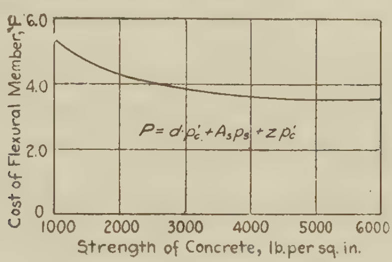

FIG. 10—RELATION BETWEEN STRENGTH OF CONCRETE AND COST OF REINFORCED CONCRETE FLEXURAL MEMBER FOR GIVEN CONDITIONS

It is noted from Fig. 10 that the relative cost of the section decreases slightly with the increase in the strength of the concrete. The decrease, however, is very small when compared with concrete in direct compression, indicating that the flexural concrete member is very little affected by the variation in the strength of the concrete. If other factors than the flexural stresses govern the size of the section, the above study is of no value in judging the economy of the member.

Since the yield-point strength of the reinforcement is fully utilized at the ultimate strength of the reinforced concrete beam or column, it becomes the criterion for the economy of the reinforcement. The economy of the reinforcement is therefore directly proportional to the ratio between the cost of the steel and its yield-point stress, that is:

$$E = \frac{p_s.}{f_s}$$

In this paper no study has been given to such important items as the reduction in dead load produced by the use of smaller sections and the increase in floor space gained by any reduction in the size of the columns. For given conditions, the economic effect of these items may be estimated directly by the use of the ordinary design formulas.

CONCLUSIONS

(1) The constant water content for concretes of a given consistency presents a basis for a rational study of the relation between the quality and the economy of the concrete.

(2) The impermeability, the durability and the fire resistance, as well as the strength of concrete are determined primarily by the cement-water ratio of the paste.

(3) The economy of a plain concrete member designed to carry a given load increases markedly with the increase in the strength of the concrete.

(4) The ordinary variations in strength qualities of the cement as well as the density of the concrete have slight effect upon the economy of the concrete.

(5) The economy of reinforced concrete columns carrying a given load increases with the increase in strength of the concrete used.

(6) If for columns having welded reinforcement, the cost of the longitudinal steel (per ton) is $0.18\frac{f_s}{f'_c}$ times the cost of the concrete (per cubic yard), the strength of the column may be increased at equal cost by the use of either steel or concrete. If the cost of the steel is greater than this ratio, the most economical column is obtained by the use of a minimum of reinforcement. For spliced reinforcement the cost ratio is about $0.15\frac{f_s}{f'_c}$.

(7) When the flexural stresses govern the size of a reinforced concrete member, the economy of the member is only slightly affected by the strength of the concrete used.

(8) The economy of the reinforcement is directly proportional to the ratio between the cost of the steel and its yield-point stress.

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Readers are referred to the JOURNAL *for October (Vol. 30), for discussion which may develop. Such discussion should reach the Secretary by August 1, 1933.*

Volumetric Changes In Neat Cements and Mortars*

BY R. E. MILLS†

Looking back over the pages of the many technical publications dealing with cements, mortars and concretes it is evident that much of the effort has been directed toward strength determinations of one kind or another. However, during the last 10 or 12 years a large number of comprehensive investigations have been carried on in an effort to evaluate the many other complex phenomena involved in cement mixtures. One of the important characteristics of cements, mortars and concretes is the change of volume as variations in temperature and moisture occur.

1. VOLUME CHANGES IN NEAT CEMENTS

In an effort to study the volumetric changes in different brands of portland cements, a comprehensive series of long-time tests was begun in the Testing Materials Laboratory of Purdue University in the summer of 1924. The specimens have been continuously under observation since that time and have involved approximately 3650 change of volume observations and about 1825 weight determinations.

Progress reports of these tests have been made from time to time in various technical publications, the last account appeared in the *Proceedings* of the American Concrete Institute, Vol. 25, 1929.

The measurements of change of volume were taken on small neat portland cement beams (2 x 2 x 24 in.) made from eight different brands of cement. A record of the physical properties of the cements is shown in Table 1. These physical tests were conducted in accordance with A. S. T. M. specifications (C9-20).

The project was divided into two series of tests. In the first series, the specimens were exposed to the natural atmospheric conditions of the laboratory following their curing period. In the second series, the specimens were allowed to dry out in the air of the laboratory for a

*Presented at the 28th Annual Convention, Washington, D. C., March 1, 1932, and since then revised to January 1, 1933.

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TABLE 1—PHYSICAL PROPERTIES OF CEMENTS

Cement	Curve No.	Percentage H_2O normal consistency	Percentage retained on 200 Mesh Sieve	Time of Setting		Soundness	Tensile strength p. s. i.	
				Initial	Final		7-day	28-day
(a) Cements shown in Fig. 1								
A	1	24	17.5	4 hr. 00 min.	5 hr. 55 min.	O. K.	250	318
B	2	24	21.0	4 hr. 30 min.	6 hr. 30 min.	O. K.	200	307
C	3	22	22.0	2 hr. 45 min.	5 hr. 10 min.	O. K.	197	303
D	4	23	19.6	2 hr. 35 min.	5 hr. 20 min.	O. K.	231	319
A, B, C, D, E, F.	5	23	18.6	3 hr. 20 min.	5 hr. 35 min.	O. K.	226	311
E	6	23	16.0	3 hr. 50 min.	5 hr. 35 min.	O. K.	199	306
F	7	25	17.1	3 hr. 35 min.	5 hr. 15 min.	O. K.	220	326
(b) Cements shown in Fig. 3								
A	1	22	18.1	4 hr. 10 min.	6 hr. 00 min.	O. K.	251	315
D	2	25	18.2	2 hr. 40 min.	5 hr. 50 min.	O. K.	242	325
F	3	23	13.7	3 hr. 30 min.	5 hr. 50 min.	O. K.	218	332
H	4	24	13.4	3 hr. 20 min.	5 hr. 45 min.	O. K.	209	311
M	5	24	11.0	2 hr. 30 min.	4 hr. 15 min.	O. K.	330	392

period of 222 days following their curing period after which they were immersed in water in which condition they have remained to date.

All of the values of change of volume shown in the several charts are due to changes in moisture content only. The temperatures were kept constant, at the time of observations, to within 10 per cent of the mean (70°F.). The measurements were then corrected for these variations with reference to temperature observations on duplicate specimens.

Fig. 1 and 2 show respectively the change of length and change of weight of neat cement beams made from six different brands of cement. The beams have been exposed to the air of the laboratory for a period of about 8 years subsequent to 10 days initial curing in water. The initial readings were taken 20 hr. after molding. However, the specimens were kept in a moist atmosphere until the time of observation.

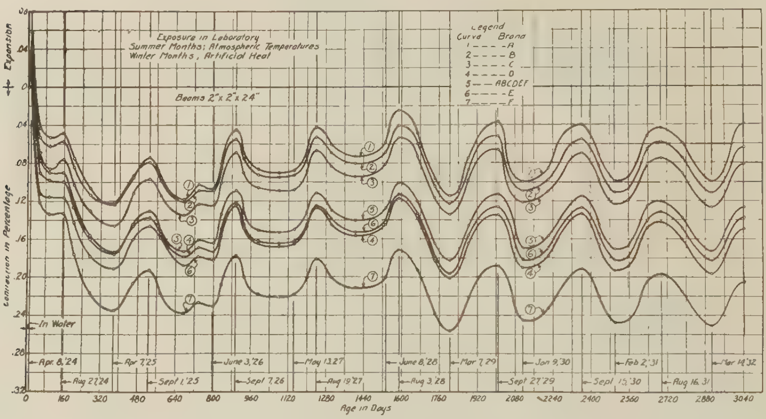

FIG. 1—CONTRACTION OF NEAT CEMENT BEAMS EXPOSED TO AIR OF LABORATORY

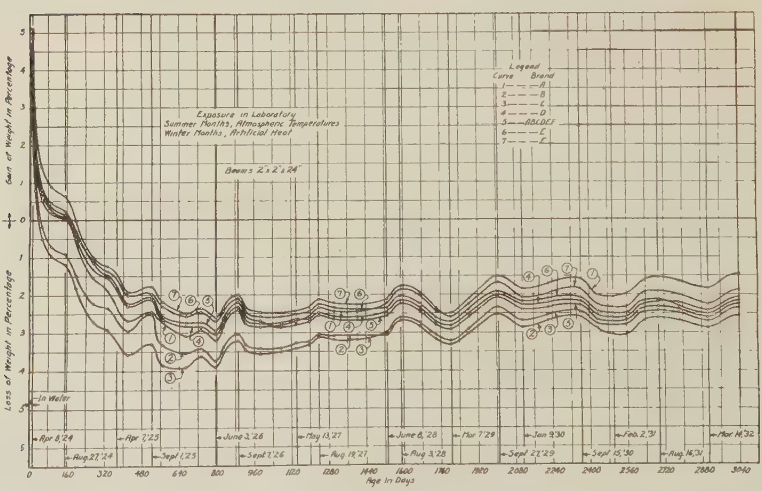

FIG. 2—CHANGE OF WEIGHT OF NEAT CEMENT BEAMS EXPOSED TO AIR OF LABORATORY

It appears from the curves (Fig. 1) that the individual behavior of the several brands of cement is confined to the first period of about 180 days, after which the movements of the several brands are approximately alike. The maximum contraction occurred during the early spring of 1929. At this time the contractions were from about 0.12 per cent for brand A, to about 0.26 per cent for brand F, giving a range of approximately 0.14 per cent for the various brands of cement.

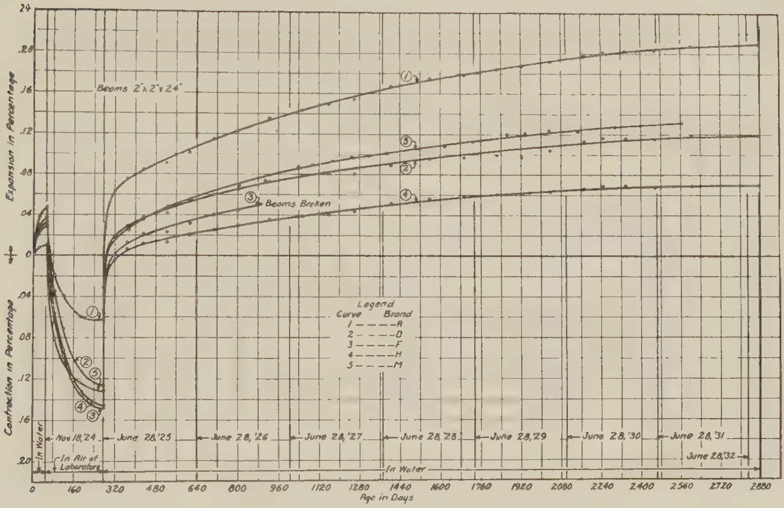

FIG. 3—EXPANSION AND CONTRACTION OF NEAT CEMENT BEAMS

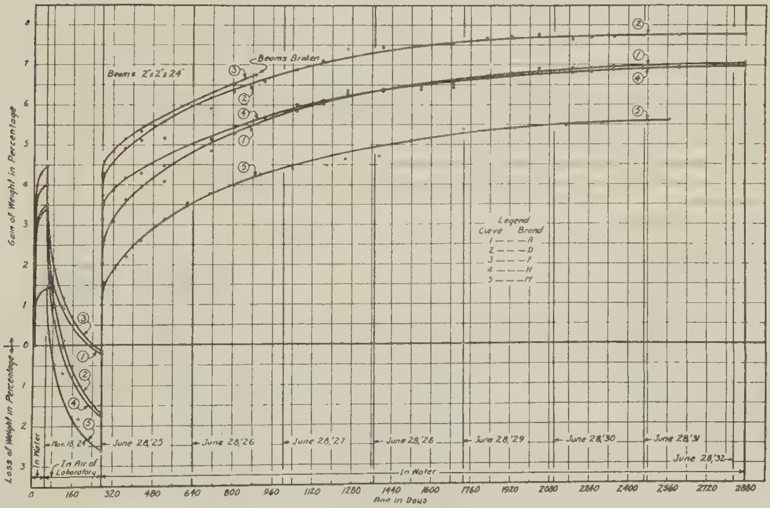

FIG. 4—CHANGE OF WEIGHT OF NEAT CEMENT BEAMS

It is of importance to note that brand F had reached a total contraction of more than twice that of brand A. The beams follow very closely the seasonal changes of the year, showing contraction in the warm, dry air of the Laboratory during the winter months, and expansion in the moist air and atmospheric temperatures of the summer months. The total amount of change of length during the year ranges from about 0.04 per cent to about 0.08 per cent for the different brands of cement.

A close inspection of the individual curves indicates a small amount of permanent expansion, or growth, to be taking place for all brands except D and F. All of the remaining curves have shifted a small amount, approximately 0.02 per cent during the period of observation.

Figs. 3 and 4 show respectively the change of length and change of weight of neat cement beams made from five different brands of cement, purchased from the market six months later than those shown in Fig. 1 and 2. It is of note that the brands of cement common to both Fig. 1 and 3 maintain their relative position.

The specimens in this series were allowed to cure for a period of 50 days in water, after which they were subjected to the drying air of the laboratory for 223 days. By the end of this drying period their contraction was from about 0.06 per cent for brand A, to about 0.15 per cent for brand F, giving a range of approximately 0.09 per cent for the several brands of cement. The specimens were then immersed in water and their expansion observed. They have now been in water for a period of about 7 years and still indicate a tendency toward expansion. The total expansion now ranges from about 0.21 per cent for brand H, to about 0.27 per cent for brand A giving a range of approximately 0.06 per cent for the several brands of cement.

2. VOLUME CHANGES IN MORTARS

A somewhat recent addition to the program of volume change observations has been the inclusion of eight masonry mortars. The method of molding specimens, the scheme for taking observations, etc., was similar to those used in the neat cement tests. The specimens consisted of small beams (2 x 2 x 24 in.).

The cements used in the different masonry mortars included several of the patent masonry cements common in the local territory, together with different compounds of hydrated lime and portland cement. In order to form a common basis of comparison a standard portland cement mortar was included with the tests.

In this project the general term "cement" refers to the cementing material used in combination with a sand in the production of a mortar.

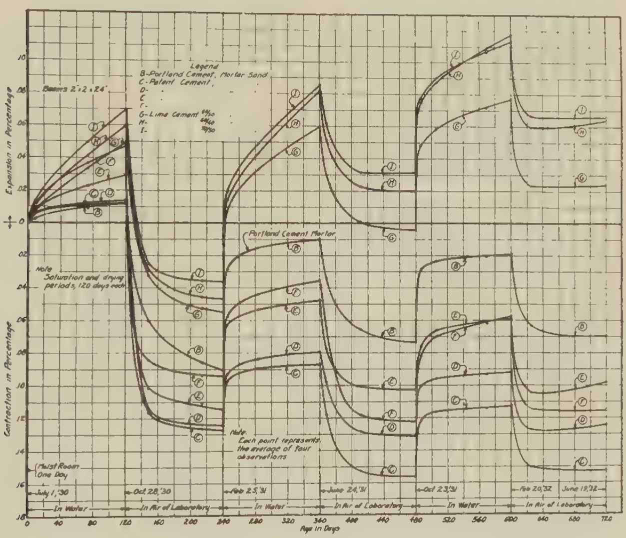

FIG. 5—EXPANSION AND CONTRACTION OF MASONRY MORTAR BEAMS

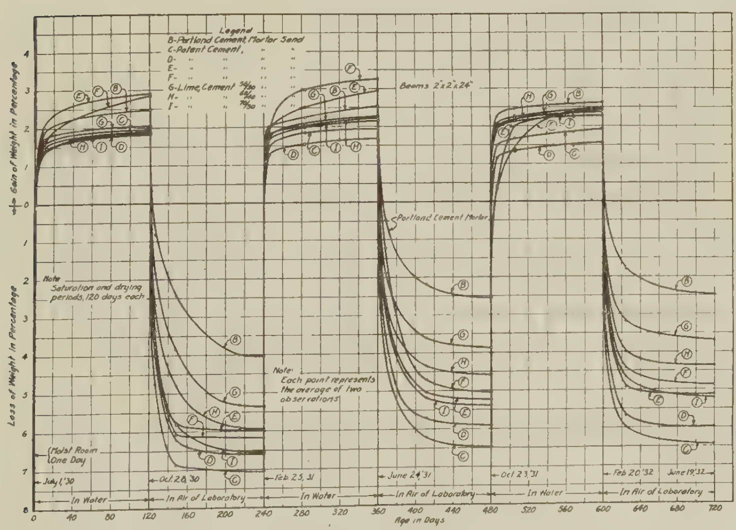

FIG. 6—CHANGE OF WEIGHT OF MASONRY MORTAR BEAMS

The term "portland cement" refers to the product as manufactured and commonly known as cement. "Patent cement" refers to a manufactured material distributed on the market under some trade name and used specifically in masonry mortars. Some of these patent cements contain portland cement and finely pulverized admixtures. "Lime cement" refers to addition of hydrated lime to portland cement to form the cementing material. In the mortars "mortar" refers to the combination of the cementing material and sand when mixed with water. All mortars, except those containing hydrated lime, were mixed 1:3 by volume. For those containing hydrated lime the proportions of portland cement, lime and sand, were G 1:1:4, H 1:1.5:5 and I 1:2.3:6.7.

Fig. 5 and 6 show respectively the change of length and change of weight of the different masonry mortar beams when intermittently wet and dry. The periods of exposure were for 120 days each, water immersion and laboratory air. An inspection of the curves (Fig. 5) shows that for successive periods of wetting and drying the four patent cement mortars C, D, E and F show considerable less expansion than they do contraction. This would indicate that for long periods of wetting, followed by long periods of drying, the total expansion is less than the total contraction, leaving a residual shrinkage. In the case of the portland cement mortar B only a very small amount of residual shrinkage is indicated. The three lime cement mortars G, H and I show an accumulative expansion, or growth, to be taking place with the successive periods of wetting and drying. This growth seems to be somewhat proportional to the lime content.

Observations of the specimens at the end of 720 days (3 complete cycles) give a definite order of total volume change from the initial observations as follows: all patent cement mortars show contraction, ranging from 0.098 per cent for mortar E to 0.152 per cent for mortar C, the portland cement mortar B shows a contraction of 0.069 per cent, while all lime cement mortars show an expansion, ranging from 0.024 per cent for mortar G to 0.066 per cent for mortar I.

Readers are referred to the JOURNAL *for October (Vol. 30), for discussion which may develop. Such discussion should reach the Secretary by August 1, 1933.*

Technological Developments in Fireproof Concrete Homes*

BY W. D. M. ALLAN AND R. E. COPELAND†

The field of concrete house construction affords a wide opportunity for engineering ingenuity to reduce the national cost of housing. Concrete is a logical material for the production of homes that are durable and fire-safe; qualities always desirable but are now increasingly appreciated as fundamental requirements for low housing costs.

Concrete construction was first used for homes about 75 years ago and remained relatively unique until World War shortage of housing and lumber literally compelled its use. With this impetus, scores of construction systems were devised and the possibilities of the concrete house were viewed in a new light. Thousands of concrete homes were built, most of them performing satisfactorily today.

Concrete was used for a considerable number of industrial housing projects. Typical examples are houses of the Cambria Steel Company at Johnstown, Pa., (Fig. 1) and group houses erected near Wilmington, Delaware, for the General Chemical Co. (Fig. 2).

By process of elimination, the less economical and adaptable systems were short lived. The major surviving type is concrete masonry, so well known and widely used that its mention only is necessary. However, several of the other systems are still being used as originally developed or with variations and have afforded the background for the technological developments of more recent times.

The different systems may be classified into four basic types—masonry, monolithic, large precast unit and structural frame. The first two should require no definition.

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*Presented at 29th Annual Convention, Chicago, Feb. 21-23, 1933. The revival of interest in the economic possibilities of concrete houses adds fresh interest to housing projects, which, as the authors of this paper have recorded, had considerable attention in and shortly after the late great war period. Since new undertakings may well be based on the record of previous successes and failures (failures due no less perhaps to unreadiness of the times than to inherent weaknesses of the systems) the present authors have prepared at the Editor's request a "Selected bibliography on concrete house building methods" with more than 60 references to articles published since January 1917—most of them in the first third of the 15-year period. This bibliography, with a partial list of concrete house building systems and names and addresses of originators or promoters, will be sent to any interested reader on receipt by the Institute of ten cents in postage stamps—Editor.

†Respectively, Director of Promotion and Technical Engineer, Portland Cement Association.

FIG. 1—CONCRETE HOUSES FOR CAMBRIA STEEL CO. EMPLOYEES, JOHNSTOWN, PA.

FIG. 2—CONCRETE GROUP HOUSING ERECTED FOR GENERAL CHEMICAL CO., WILMINGTON, DA.

The large precast unit method includes those systems which utilize precast slabs of considerable size for walls, floors and roofs. Pitched roofs generally are of wood covered with an approved roofing.

The structural frame method utilizes precast or cast-in-place columns, beams and joists supporting non-bearing filler walls and partitions. The roof may be of precast concrete members and fire-resistant sheathing and roofing or of conventional wood construction.

Technological developments may be broadly divided into (a) refinements of conventional methods, and (b) new variations of unusual methods.

REFINEMENTS OF CONVENTIONAL METHODS

Lightweight Concretes—Of far-reaching importance has been the rapid development of lightweight aggregates as obtained from burned shale, blast furnace slag, pumice and cinders. A number of such aggregates have recently been introduced commercially and experimentation on new processes is being continued. Commercial avail-

ability has been extended to reach the more populous and active sections of the country.

Lightweight aggregate concretes have opened up new fields for precast products. The reduced unit weight of the concrete increases the handable size of masonry units, increasing the mason's output and lowering masonry costs. The superior insulating, nailing and sound absorbing qualities can be combined to produce masonry units and precast slabs incorporating strength, insulation, weather-tightness, exterior and interior finish and acoustical correction.

FIG. 3—PAINTED CONCRETE ASHLAR EXTERIOR WALLS

Expanded concrete such as Aerocrete also has excellent possibilities for house construction as a lightweight insulating material. The Chicago architects, Bowman Brothers, decided on 3-in. Aerocrete insulating wall slabs for their proposed housing scheme.

Concrete Ashlar—Painted concrete ashlar is interesting as an architectural refinement in masonry construction and also because it is economical. (Fig. 3.) A chief saving is in the exterior finish, consisting of two coats of portland cement paint at about 3½ cents per sq. ft. More recently concrete ashlar has been adapted for interiors (Fig. 4) in which two-tone color effects are obtained by using a lighter shade for the final coat of paint applied only on the high spots of the open-textured units.

Acoustical Correction—Homes generally have sufficient sound absorption in rugs and cloth covered chairs to effect a fairly quiet

condition. However, this condition is improved with walls of the open-textured, porous type masonry, painted or unpainted, which deaden sound more quickly.

Coloring Treatments—New information on coloring pigments, paints and stains for concrete has been developed. Committee 408 of this Institute has formulated recommendations of practical value to the contractor and products men in obtaining satisfactory color work.* The surface treatment of precast products in the plant is a forward step in producing the economical wall and floor unit combining structural, decorative and insulating properties. The St. John's Church, Milwaukee, and the Wallkill Prison, New York State, are examples of the use of masonry units given an exterior treatment at the plant.

FIG. 4—INTERIOR WALLS OF CONCRETE ASHLAR PAINTED A TWO-TONE COLOR EFFECT

Reinforced Concrete Masonry—Some engineers designing structures for earthquake and hurricane zones believe that masonry houses should be reinforced to increase their resistance to lateral forces. This is readily done as shown in Fig. 5. Vertical rods are inserted in core spaces which are later filled with mortar. Reinforced girts or belt courses extend around the exterior walls and into the partitions. It is important that the various structural elements be securely tied together. Additional lateral strength is provided by the reinforced concrete floors. The size and spacing of the wall reinforcement can be varied to meet the theoretical requirements as determined for a particular structure and the assumed wind or seismic forces.

*"The Use of Color in Concrete," progress report, Am. Concrete Inst. JOURNAL, April, 1931, *Proceedings*, Vol. 27, p. 975.

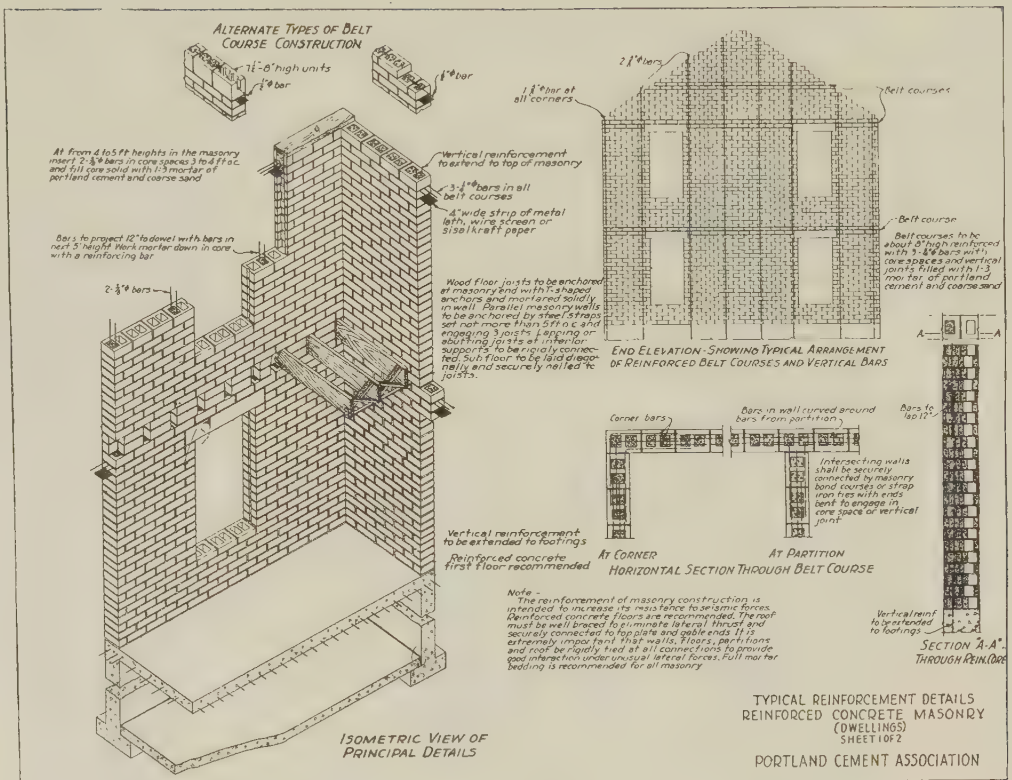

FIG. 5—DETAIL OF REINFORCED CONCRETE MASONRY CONSTRUCTION

Precast Floor Joists—The recent commercial production of precast concrete floor joists provides an economical, fire-resistive type of floor especially well suited for dwellings. Fig. 6 shows typical details. When made with lightweight aggregates, the joists weigh about 13 pounds per linear foot for residence loads and spans and can be handled by two men, in lengths up to 18 ft. They are set on the bearing walls or other supports about 24 in. on center and slab forms, such as metal pans, metal lath or loose planks on spreaders, are supported by the joists and a 2-in. slab is placed. The slab and joists are securely bonded in one or more ways so that they act together in resisting bending stresses.

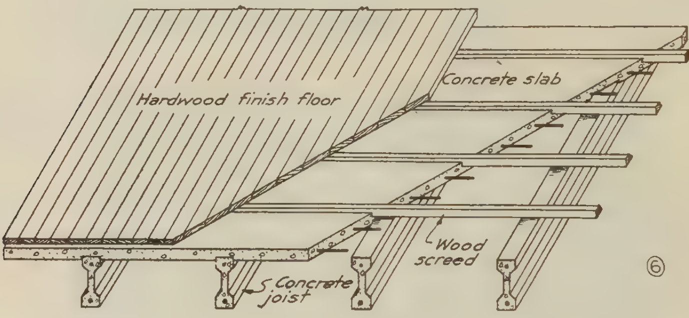

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FIG. 6—LITH-I-BAR FLOOR

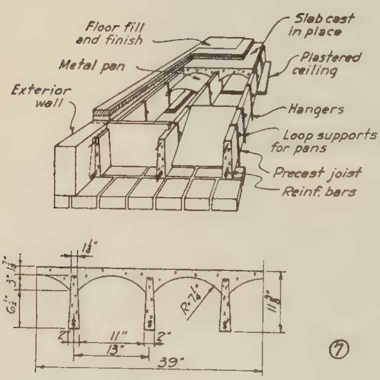

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FIG. 7—(LEFT) ISTEG CONCRETE FLOOR

Two plants make a patented precast joist known as Lith-I-Bar, one in Kalamazoo, another in Philadelphia. The process includes a high production machine making the joists on pallets drawn under a series of heavy rollers to compact the concrete. The same company is experimenting with a precast concrete plank about 8 ft. long by 12 in. wide and 2 in. thick, to be used on top of the joists. The bond is made with portland cement mortar. It is claimed that this combination will

provide a low-cost, fire-resistive floor, rapidly constructed with unskilled labor.

Fig. 7 shows the Isteg precast joist, in combination with a cast-in-place slab, already used for more than 500,000 square meters of floors in Austria and Germany. Its introduction into England and the United States is now being negotiated. As with other precast joist types, its principal economy is in eliminating much of the form work required for standard monolithic construction.

Junior beam-concrete slab floors also are suitable for residences and have been used in more than 3,000 dwellings. To be considered fire-safe or fireproof, the beams should be protected by a plaster ceiling or concrete covering.

NEW VARIATIONS OF UNUSUAL METHODS

Several industries and individuals are giving particular attention to the more complete prefabrication of the elements entering into house construction, it being contended that factory methods are more efficient and labor cheaper than on the job. Like principles have motivated the development of precast concrete unit construction, essentially a prefabrication method. Among its first exponents were Grosvenor Atterbury, John Simpson and the Unit Construction Co. At Forest Hills, Long Island, under Mr. Atterbury's supervision, large units required heavy handling and transportation equipment. Similar methods were employed on the Unit construction job at Youngstown, Ohio.

Considerable thought now is being given to the development of more adaptable precast types utilizing slabs about 3½ or 4 ft. wide, of story height or span length. Fig. 8 shows the use of medium size precast units as suggested by John E. Conzelman, formerly chief engineer of the Unit Construction Co. He estimates that modern firesafe 4 and 5-room, one story cottages can be built for $2500 to $3500 by this method.

The Concrete Housing Corp. of America, which operated in New York City, used a similar type of wall slab and reports building nearly 2,000 small cottages and garages.

A variation of this method has been used by Swan House, Inc., in houses at Dolton, Ill., and recently at Wheaton. One of the Dolton houses is shown by Fig. 9. The exterior walls are of precast posts about 4 feet apart, grooved for 1½-in. thick slabs, which are precast with a special facing mixture. Fig. 10 shows the principal details—the post and outside details and the 3-in. insulating blocks fitted between the

posts on the inside and the insulation board to serve as a plaster base on furring strips nailed to wood inserts. Steel trusses, cement asbestos sheathing and shingles compose the roof. Floors are of bar joist, cement asbestos sub-floor and hardwood flooring.

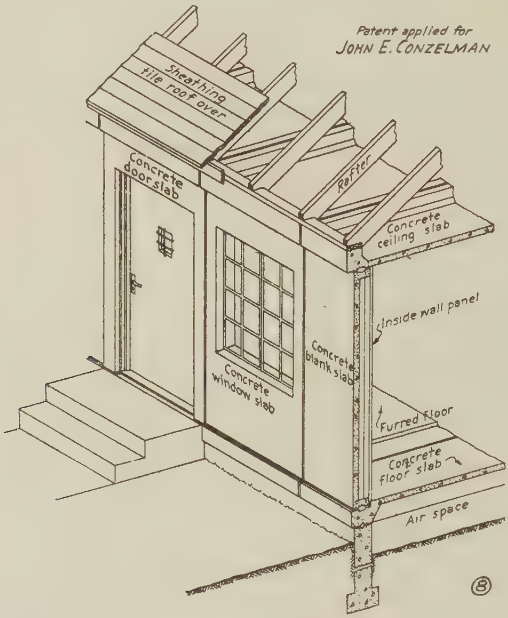

FIG. 8—CONZELMAN UNIT CONSTRUCTION

Swan House, Inc., is continuing its studies toward simplification and reduced costs.

W. C. Broughton, Kansas City, Mo., has built more than 75 dwellings and 300 garages, some within the last two years, with a similar precast unit construction.

B. V. Zamore, Toledo, Ohio, has recently completed a one-room experimental precast unit structure with wall and floor slabs of ribbed section. Metal stripping is applied over the joints between the slabs. The inside is finished with large sheets of rigid insulation, inexpensively painted or plastered with special materials. Mr. Zamore is one of the first to recognize that joints between slabs could be made watertight and the exterior appearance improved by accentuating the joint lines with a metal strip.

From the technological viewpoint, the precast unit type of construction is much more feasible today than when introduced 15

FIG. 9—PRECAST CONCRETE UNIT TYPE COTTAGE BUILT BY SWAN HOUSE, INC.

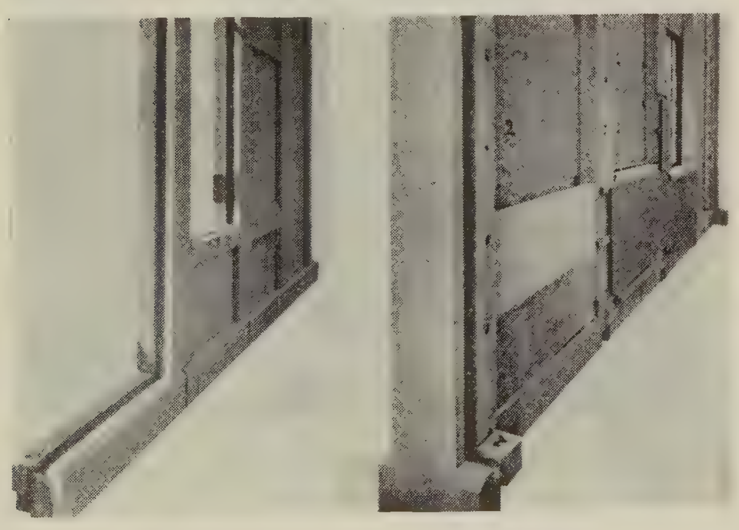

FIG. 10—CONSTRUCTION DETAILS, SWAN HOUSE SYSTEM WALL EXTERIOR AT LEFT; INTERIOR AT RIGHT

years ago. Improvements in lightweight aggregates and their use have provided a lightweight structural and insulating concrete. The vibration method of placing concrete is particularly applicable to precast slabs. Architecturally, the method is best suited to the simpler architecture—not a handicap where low-cost housing is the primary object.

FIG. 11—MONOLITHIC CONCRETE HOME OF MRS. FRED DINSE, PARK RIDGE, ILL.

FIG. 12—PANELLED EFFECT WITH BROWN STAINED CONCRETE, DINSE HOME

The successful production of lightweight roof slabs up to 8 ft. lengths indicates that the manufacturing problems could be solved readily with a volume of production.

Monolithic—New form materials such as Plywood and Presdwood sheets should be included among the important developments in monolithic construction. The residence shown by Fig. 11, built in 1931 for Mrs. Fred Dinse, Park Ridge, Ill., is practically 100 per cent

monolithic concrete with large Plywood sheets used for forms where smooth surfaces were desired. The exterior walls, floors and roof were effectually insulated by air spaces and insulating material as indicated by a wall cross-section in Fig. 13. No plaster was used, the exterior and interior surfaces being decorated directly. Fig. 12 shows an effect of wood panelling obtained by the use of rough form lumber and the application of a brown stain on the concrete.

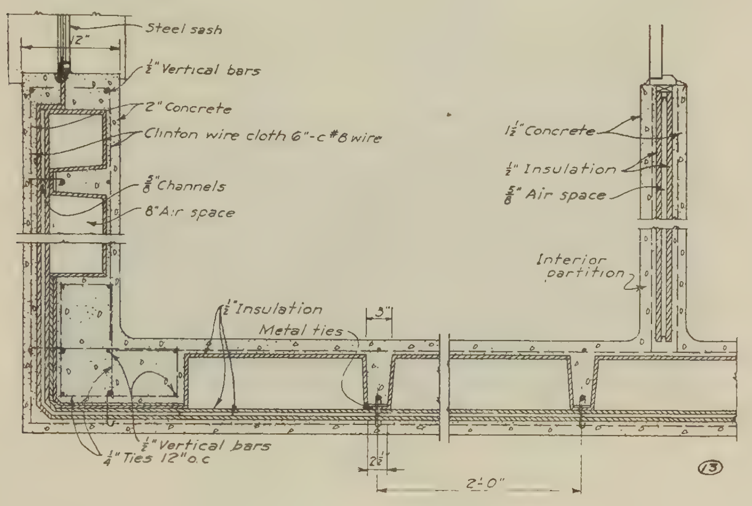

FIG. 13—TYPICAL WALL PLAN OF INSULATED CONSTRUCTION

Hollow Double Walls—Monolithic and precast concrete unit houses are probably more common in England than in this country. In the construction of monolithic concrete houses at Suffolk, England, sliding forms, of heavy wood planks, were raised by the lifting devices about five inches per hour (Fig. 14). The hollow, double walls have an outer shell of dense stone concrete for watertightness and an inner shell of breeze concrete for insulation. Five-room cottages on one-quarter acre of land sold for 380 pounds sterling. The exterior and interior walls were erected in 18 working hours with a 16-man crew. This rate of progress indicates that the method is both rapid and economical. The construction schedule called for the completion of a house two weeks after the walls were started.

Several thousand dwellings with hollow double walls have been built in this country by various methods and devices, of which the Van Guilder mold was probably the most extensively used. Its operation

consists essentially of tamping concrete solidly into the mold, loosening the side plates by a lever arrangement and moving the mold forward one length for the next set-up. Charles R. Lehrack of Los Angeles has recently developed a lightweight movable mold which is being considered for use on a proposed 50-house project on the West Coast.

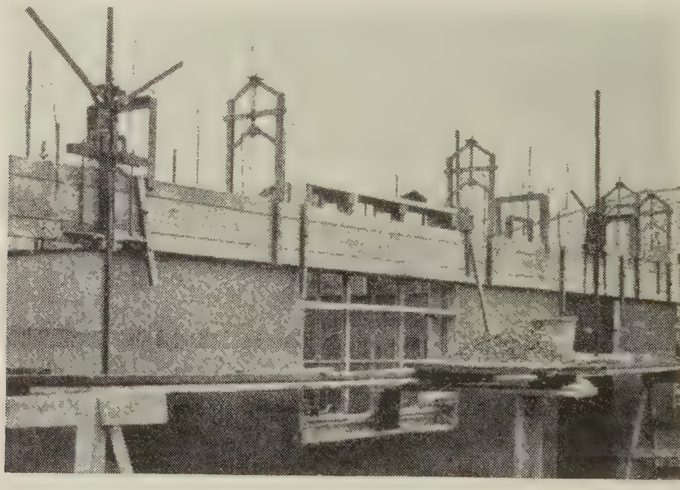

FIG. 14—SLIDING FORMS USED IN BUILDING CONCRETE COTTAGES, ENGLAND

It would seem that the hollow double wall is well adapted to the combination of a dense concrete outer shell and a porous insulating inner shell, with the plaster applied directly or omitted. With the continuous air space, the insulation value would be satisfactory for the low-cost type house.

The former view that monolithic concrete is economically adapted only to large housing developments is gradually changing because of the developments in design and construction technique. For instance, the provision of adequate insulation has been simplified by building the insulation into the wall, either by special design of the section or by insulating concretes or special materials such as corkboard and Masonite. It is the confident opinion of some that monolithic concrete can be used with practically the same efficiency on a group of two or three dwellings as on the large project.

Structural Frame—Typical recent examples of the structural frame method include houses built by A. H. Olmsted at Rye, N. Y., and by the Brice-Pearson Co. at Niles Center, Ill., and other Chicago suburbs.

A special feature of the Olmsted system is the precast wall studs set 16 inches on center as shown by Fig. 15. A wood nailing strip is secured to the edge of the stud at the time of casting for nailing on the lath or plaster board. The insulating board is arched between the studs to serve as inside forms. The hollow space in the wall adds to its insulation value and accommodates conduits and electrical wiring. Practically any type of exterior treatment can be applied to the structural wall. Stone or brick facing is anchored by means of the form tie wires. Wood shingles or clapboard are nailed to wood strips attached to the outside edge of the stud. The houses have reinforced concrete floors and are thoroughly high-grade in every respect.

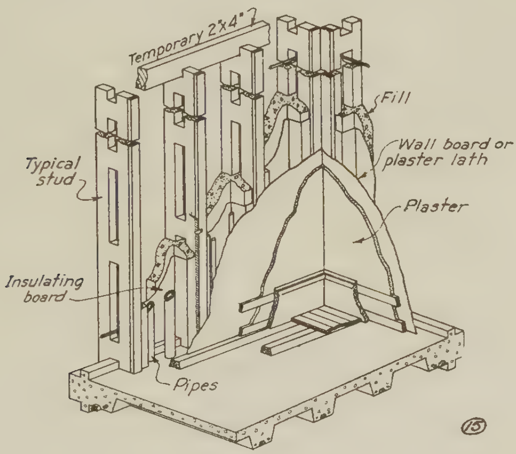

FIG. 15—OLMSTED SYSTEM

The Brice-Pearson system utilizes precast reinforced concrete columns and beams in conjunction with non-bearing and bearing masonry walls. Floor slabs are precast channel-shaped units about 16 inches wide and of span length. This company manufactured most of the units in a factory and trucked them to the job. While the economy of the construction is open to question, it is considered a contribution to the field because the problems of manufacture, transportation and erection were solved to the extent that a number of residences and apartment buildings were actually constructed.

The structural frame method in which the columns, beams and joists are cast in place was the basis of the Donaldson and other systems used for several hundred houses a number of years ago. Variations of these systems are still in use. R. A. Sandberg, a Rock

Island architect, recently designed and built a two-story house having two-way reinforced concrete floors and roof supported on 10 or 12 reinforced concrete columns. It is reported to be an excellent example of firesafe construction. The exterior walls were built of 3-in. cork block with 1-in. reinforced portland cement stucco on both sides.

THE CONCRETE HOUSE OF THE FUTURE

It is probable that the concrete masonry type will continue to predominate and that there will be an increased use of fire-resistive floors, particularly as the low-cost types employing precast joists become more widely available. The use of concrete ashlar for exterior and interior walls will become more common and will tend to eliminate costly plastering and exterior finishes and result in lower building costs.

Future developments in the monolithic and precast unit types will depend a great deal on future building practices. If people accept the standardized or semi-standardized architecture, such as is being experimented with, the logical development will be the prefabrication in factories of concrete wall and floor slabs having all of the essential requirements of structural strength, weather-tightness, insulation and interior and exterior finishes.

It is believed that the cost of concrete house construction will be gradually reduced by an evolutionary process involving improvements in construction methods and refinements in materials. A new machine is now being developed for producing a watertight surface on porous insulating concrete, and it is conceivable that within the next few years the concrete products industry will be producing masonry units which will provide a finished wall, even to the inclusion of the desired color and textural effect.

Readers are referred to the JOURNAL *for October* (*Vol. 30*), *for discussion which may develop. Such discussion should reach the Secretary by August 1, 1933.*

Compaction of Concrete Through the Use of Vibratory Tampers*

BY RAYMOND E. DAVIS AND HARMER E. DAVIS†

INTRODUCTION

It is significant that, though practically all manufacturing processes are highly mechanized, we are placing and compacting concrete very largely by hand methods, much the same as in the days before the invention of the concrete mixer. Here lies the greatest opportunity for advance in concrete construction. The mechanically operated vibratory tamper, of which several varieties are commercially available, provides an efficient means both of reducing costs and improving the quality of concrete. During the last year in particular, the vibratory tamper has attracted the attention of engineers and contractors in this country, and it has demonstrated its usefulness under a wide variety of conditions.

This paper describes investigations in progress in the Engineering Materials Laboratory of the University of California to determine the relative advantages of vibratory tamping compared with hand tamping. These studies include tests to determine:

(a) Ease of placement
(b) Form pressures
(c) Homogeneity of concrete
(d) Bond of new concrete to old
(e) Strength
(f) Density
(g) Durability
(h) Volume changes

Further studies are in progress to determine the most satisfactory gradation of aggregates for concretes placed by vibratory methods and to determine the most effective means of placement and compaction through the use of the vibrator.

For these investigations two types of compacting apparatus have been employed, hereinafter designated as (a) the internal vibratory tamper and (b) the external vibratory tamper.

*Presented at 29th Annual Convention, Chicago, Feb. 21-23, 1933.
†Respectively Professor of and Instructor in Civil Engineering, University of California.

The internal vibrator consists essentially of a vibratory element connected to either an electric or an air motor, the element being a hollow tube within which rotates a shaft with attached eccentric weights. In use, the vibratory element is inserted and withdrawn from the freshly deposited concrete. During this process the whole element vibrates with a frequency equal to the speed of the motor, ranging from 4000 to 7000 r.p.m., depending upon the details of the design. As a result, vibratory impulses are transmitted in all directions to the surrounding concrete. For the machines used in these tests, the vibratory elements, which were connected either rigidly or through a flexible shaft to the driving motors, were approximately 30 in. long and of diameters, 1, 1¾, and 2⅜ in.

The external vibrator consists of an electric motor with an eccentric at one end of its shaft, the housing of which unit is attached either to a float or to a vise. In operation, the float is applied to the free surface of the concrete, or the vise is clamped to the forms. The vibratory impulses, produced by the eccentric on the rotor, which is driven at a speed of approximately 3500 r.p.m., are transmitted either through the float or through the forms to the concrete surfaces.

Acknowledgment is made of the valuable services of Messrs. P. K. Davis, John S. Hamilton, Loren W. Hunt, and R. W. Spencer, all of whom have been engaged in carrying out the details of the tests herein reported.

OUTLINE OF TESTS

Inasfar as possible the tests were designed to simulate conditions encountered in construction practice, with the added factor of close control through careful laboratory procedure. The specimens were: (a) cylinders ranging from 6 by 12-in. to 2 by 3 ft., (b) wall sections varying from 2 ft. by 2 ft. by 6 in. to 2 ft. by 7 ft. by 6 in., (c) columns up to 9½ ft. high, (d) plain concrete beams 4 in. by 6 in. by 24 in., and (e) volume change bars 3 in. by 3 in. by 40 inches.

(In addition, some of the form-pressure tests were carried out during the construction of a structure having walls 6 in. thick).

The mixes employed in the investigation ranged from those used in ordinary building construction (1:2-½:3½ with maximum size aggregate ¾ in. to 1½in.) to that used in mass concrete construction (1 bbl. of cement per cu. yd. with maximum size aggregate 6 in.). The consistencies ranged from slumps of approximately ½ in. to 8 in.

The coarse aggregate for the major portion of the tests was gravel. but for the gradation studies both crushed stone and gravel were employed. For the major portion of the tests, the fine aggregate was a

natural sand of maximum size ¼ in. and fineness modulus of approxmately 3.0. For certain of the tests, to secure desired gradation of the fine aggregate, small quantities of river sand (fineness modulus 2.0) were added. To obtain desired and uniform gradations, both fine and coarse aggregates were screened into several sizes and recombined.

In all cases except those noted, the specimens were moist-cured at 70°F. and the strength tests were made at an age of 28 days.*

CONCLUSIONS

1. The indications are that under proper conditions as regards forms and methods of depositing fresh concrete therein, mechanical tamping or compaction through the use of the vibratory tamper is in every respect superior to hand-tamping.

2. Regardless of the quality of the concrete, the effect of vibratory tamping is universally to increase the unit weight over that obtained by hand-tamping, but the manner of tamping has no appreciable effect upon the strength, indicating that within the range of mixes which it is possible thoroughly to compact by hand, the water-cement-ratio-strength relation holds for vibrated concrete as well as for hand-tamped concrete.

3. During the period of vibratory tamping, the pressure of fresh concrete against the form, for that portion of the mass which is in agitation, is substantially the same as that which would be produced by a liquid having the same density as the concrete. This is true regardless of the consistency of the mix. After the cessation of vibration, however, the pressure against the forms drops off rapidly, until in a few minutes it is approximately the same as that produced by hand-tamped concrete under ordinary conditions. Under conditions encountered in building construction, with the limitation there obtained as regards rate of pour and mass of concrete to be vibrated, the maxi-

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*In keeping with a new publication policy, omission is here made of parts of this paper which will be of considerable interest and importance to those concerned with the details of the test procedure in the various phases of the investigation reported or with the analyses of the details of the test results upon which the "Conclusions" are based. A copy of the complete paper including tabulated data will be sent to any Institute member on receipt of 50 cents—to any non-member on receipt of $1.00. The omissions from the paper include descriptions of test procedure and results under the headings which follow. In writing please ask for *complete* paper by title.—EDITOR

Relative Placeability of Hand Tamped and Internally Vibrated Concretes
Pressure of Fresh Concrete upon Forms
Homogeneity of Concrete
Bond of New Concrete to Old
Effect of Vibratory Tamping on Strength and Unit Weight
Effect of Ratio of Fine to Coarse Aggregate on Strength and Unit Weight
Effect of Vibratory Tamping on Durability
Table 1: Segregation Studies—Results of Compressive Tests on 15 by 24-in. Cylinders.
Table 2: Segregation Studies—Results of Compression Tests on 24 by 36-in. Cylinders
Table 3: Compression Tests of 3 ft. by 3 ft. by 6 in. Wallettes and 6 in. by 12 in. Cylinders Tamped by Vibratory and Hand Methods
Table 4: Effect of Ratio of Fine to Coarse Aggregate on Strength and Density of 15 in. by 24 in., Cylinders using Internal Vibration and Hand-Tamping.
Table 5: Comparison of Durability of Hand-Tamped and Vibrated Specimens

mum form pressures may be expected to be of the order of perhaps 3 to 4 p. s. i. .

4. Within the limited range of the experiments, it appears that much harsher mixes may be successfully compacted by vibratory tamping than by hand-tamping. This suggests the possibility of reducing the ratio of fine to coarse aggregate, and perhaps otherwise changing the grading of the aggregates for concretes which are to be placed by vibratory tamping, since there are obvious advantages, particularly in the matter of volume changes, in reducing the quantity of fine materials of the mix.

5. Durability or resistance-to-the-action-of-weather of concrete may be improved by vibration. Accelerated tests to demonstrate this property, consisting of alternations of high and low temperatures and wetting and drying, indicate that universally the strength of internally vibrated concrete is less affected by this durability treatment than the strength of corresponding hand-tamped concrete.

6. For the range of conditions investigated, there appears to be a definite advantage in favor of internal vibration of the mass as against vibration applied to the surface of the concrete. This applies particularly to the homogeneity of the resulting concrete and to the efficiency of recombination when segregation has taken place between the time of mixing and placing. Also, it appears to apply in the matter of bond between new concrete and that which has hardened, under conditions such as those which exist along construction joints, where several days may elapse between one pour and the next.

7. Perhaps the most striking advantage of vibratory tamping is the effectiveness with which concrete may be placed, particularly in intricate forms and in thin sections heavily reinforced. For consistencies which are normally employed in building construction and similar work, compaction is secured much more rapidly through the use of the vibratory tamper than by hand-tamping. For example, for one of the conditions which were investigated, it was found that the time required to place a given volume of concrete of 6 to 7-in. slump by vibration was approximately 20 seconds, while hand-tamping the same quality of concrete under the same conditions took in the neighborhood of 60 seconds. Obviously, the time required for vibratory tamping as against hand-tamping will depend upon a variety of conditions, so that the above example cannot be universally applied, but it is indicative of the general findings.

8. Further, for the same ease of placement and the time required for thorough compaction, it is possible through the use of the vibratory

tamper to place mixes of much drier consistency than those which can be placed by hand tamping. Here, for example, under the same conditions of placement as stated under (7) and with a placement period of 60 seconds, it was found possible through the use of the vibratory tamper to reduce the slump from the 6½ inches required for hand-tamping to 1 inch. It is believed that in general the concrete for walls, beams, and columns, may be effectively and easily placed through the use of the internal vibratory tamper when the slump is as small as 2 to 3 inches, and that in mass concrete construction, concrete may be effectively placed when the slump is as small as 1 inch or less.

9. From (8), it follows that through the use of the vibratory tamper there are possibilities of considerable reduction in the cement content of a mix without sacrifice of strength, by maintaining a required water-cement ratio and using the drier consistency which vibration makes possible. By way of illustration, for a group of wall sections upon which tests were made, by reducing the slump from 6 inches to 2 inches, with the water-cement ratio held constant, the cement content was reduced from 6 sacks to 5 sacks per cubic yard without any sacrifice in strength. Aside from the immediate economic advantage of reduced cost of cement, this reduction in cement content would also seem to possess the added advantage of lower volume changes in the concrete, and possibly also greater resistance to the action of weather.

10. In studying the effect of internal vibration, as against hand-tamping, upon the homogeneity of the resulting mass, artificial segregations, made under identical conditions, indicate in general that with the proper manipulation of the vibratory tamper, rock pockets may be entirely eliminated. With hand tamping, this elimination of rock pockets is in many cases impossible once segregation has taken place. In the investigations here considered, it was found that not only was more time consumed in attempting to reconsolidate the mass through hand-tamping, and not only were the hand-tamped masses in general badly honeycombed, but the strength of the imperfectly compacted hand-tamped concrete in which there had been artificial segregation was materially lower than for the internally vibrated concrete.

11. Of particular importance seem to be the results of investigations to determine the effectiveness of bond along the construction joint at the end of a day's pour. These tests indicate that without any preparation other than brushing and wetting the old surface, even with artificial segregation of the coarser materials at the bottom

of the new layer, there may be obtained through internal vibration complete bond, though several days have elapsed between pours. For all methods of placement other than internal vibration, under the same conditions, there is very little, if any, bond between the old and new concrete. The lack of bond of new concrete to old, where hand-tamping has been employed, is frequently manifest through leakage along construction joints in foundations, retaining walls, dams, and other structures which are subject to water pressure.

FUTURE INVESTIGATIONS

It is felt that the investigations here reported merely scratch the surface so far as methods of mechanical compaction are concerned, and that here lies a field for extended experimentation the results of which would be of great value to those engaged in concrete construction.

1. For the design of forms, adequate data concerning concrete pressures should be obtained for a wide variety of rates of pour, types of aggregate, and consistencies.

2. It should not be taken for granted that gradations of aggregate which are suitable for hand-placement are likewise best for mechanical compaction. In particular, studies should be made to determine the effect of gradations harsher than are now considered to be within the workable range, and there should be investigated the results of such gradations upon strength, durability, volume changes and other properties of concrete which are recognized as important.

3. The question of air bubbles, which is so troublesome in the more plastic concretes under conditions of both hand and mechanical tamping, is one deserving of some consideration, with thought given to the manner of placing the concrete, consistency, method of manipulating the vibratory tamper, and gradation of materials so as to reduce to a minimum the volume of entrained air.

DISCUSSION

*Ross White**—Based on experience with vibratory tampers on two large concrete dams, Professors Davis' conclusions as to the effectiveness and advantages of such machines are fully supported by practical use in the field.

On the Chute-a-Caron Dam[1] only the external vibratory tamper was available. On mass concrete, it was applied directly to the surface of the freshly deposited concrete by means of a float or platform. A

*Water Dept., Pasadena, Calif. Mr. White's discussion was presented by Professor Davis, supplementary to the foregoing paper.

[1]"Concreting methods at Chute-a-Caron Dam," by I. E. Burks, JOURNAL, Amer. Concrete Inst., Feb. 1930, *Proceedings*, Vol. 26, p. 315.

stiff concrete, that from a practical standpoint would have been entirely unplaceable by hand methods, was readily compacted by the machines. The concrete was placed by means of four yard bottom dump bucket, and batches of this size were quickly flattened out and thoroughly bonded to either old or new concrete below. The proof that such bonding actually was accomplished is in the fact that leakage through day's work planes is negligible on this dam, only a few insignificant wet spots having developed when the dam was put into service.

Some difficulty was had, however, in preventing honeycomb spots against the forms, especially under the overhanging downstream forms, where it was difficult to get the "platform" type vibrators into position.

The same type of vibrator was also used, clamped to the forms, on reinforced concrete powerhouse construction, but was not found to be very effective when applied in this way. On similar reinforced concrete work on the Pine Canyon Dam the internal type vibrator described by the authors has been used, with an effectiveness that is startling when first observed.

When concreting started on the Pine Canyon Dam, now under construction for the Pasadena Water Department, the Contractor provided not only platform type external vibrators, but both air driven and electric driven internal type vibrators, the latter being known on the job as "spud" vibrators.

Mass concrete with 0.95 bbl. of cement per cu. yd. and a water-cement ratio of 0.88 is being handled cheaply and effectively. Slump of this concrete is only about $\frac{3}{4}$ in., yet a honeybomb spot against the forms is almost unknown. Strength of 14 x 28 in. cylinders at 28 days is over 2000 p.s.i. and over 3000 p.s.i. at 90 days. With rather light aggregates, test blocks cut at random from the dam show an average weight of over 155 lb. per cu. ft.

With the concrete being delivered to the dam in four yard bottom dump buckets, a gang of six men and a foreman, using two "spud" internal vibrators and one platform type external vibrator will compact the above concrete at a consistent rate of 50 yd. per hour, with many days showing an eight hour average of over 60 yd. per hour.

Both types of vibrators are effective on mass concrete, but it is thought that the best results are obtained when used in conjunction. The platform type is especially useful in compacting the top of a pour, leaving it in excellent condition for brooming off and cleaning, while the spud type internal vibrator is invaluable in assuring good workmanship against forms, and in bonding one batch with another. In

confined forms of any kind, the spud type internal vibrator has been found to be much more effective than the external type.

Another point definitely established on the Pine Canyon Dam work is that rates of vibration of 4000 r.p.m. or above are much more effective than slower rates. It is estimated that a change from 3000 r.p.m. to 4000 r.p.m. nearly doubled the effectiveness of both internal and external types. 4500 r.p.m. showed still better results, but no data are available on higher speeds.

In large masses of stiff or harsh concrete, the internal type vibrators must have considerable power for efficient work. Some of this type, of only fractional horse-power are practically "frozen" when thrust into a mass of such concrete, while a vibrator of the same type, but equipped with a 3 h.p. motor will compact the same batch of concrete in a few seconds. Large power does not seem to be so essential in the case of the external type vibrators.

Experience in the field confirms the author's conclusion that the use of vibratory tampers produces high form pressures. Forms to support such pressures are easily built, however, and are a small price to pay for the great improvement in quality and appearance. For a concrete of a given strength, an actual dollars and cents saving can usually be shown if vibratory tampers are used, due to the decreased amount of cement required when the water-cement ratio is lowered. Labor costs of placing are also lower, in spite of the stiffer mix.

Readers interested in placing concrete by means of vibration are referred to other papers in this JOURNAL *and to contributions received at the 29th Annual Convention, publication of which is necessarily deferred to October 1933 (Proceedings, Vol. 30). A closure date for discussion will be announced at that time. Discussion of papers published here in part only (as indicated by footnote) will be acceptable from those who obtain the complete papers from the Institute. Contributors to discussion should in every instance send two copies of their discussions—one for the author of the paper discussed.*—EDITOR

Vibrated Concrete*

BY T. C. POWERS†

INTRODUCTION

Although vibration as a means of placing concrete has been finding favor in the field during the last few years, it has been given little systematic study—at least, few quantitative data have come to the writer's attention. The Portland Cement Association Research Laboratory has, during the last year, been making an effort to acquire basic information on vibration and its effect on concrete.

Before attempting laboratory experiments it seemed best to fortify ourselves with some first-hand observations of vibrators as they are now being used in the field. Last summer, the writer made such observations on both building and paving construction.

FIELD TESTS

One of the most interesting field studies was that made on the U. S. Government work at Davenport, Iowa, one of the units of the Mississippi River 9-Foot Channel project. On this job, internal vibrators were in use. Through the courtesy of the U. S. Government engineers and S. A. Healy, the contractor, it was possible to make direct comparisons between hand placing and vibration. Conditions were such that the two methods could be compared under exactly parallel placing conditions with excellent control of variables. These tests showed that by appropriate changes in consistency and design of the mixture, it was possible to reduce the water content of the concrete more than a gallon per sack of cement; or it was possible to produce concrete of a given water-cement ratio with fully a sack of cement per cubic yard less than that required when placing by hand. The changes in mixture composition consisted essentially in reducing the proportion of sand and increasing the coarse aggregate. As will be pointed out below, vibration imparts no new properties to concrete.

*In its preliminary form this paper was presented at the 29th Annual Convention, Chicago, February 21, 1933. As here published only the outstanding developments are recorded, chiefly with a view to the interests of those who are concerned with results which may be expected from the use of vibration in placing concrete. The complete record of the investigation will consist of approximately 12 pages of tabulated data, 16 figures, 12 pages (single space) typewritten sheets. This complete record will be sent to any Institute member on receipt by the Institute of 75 cents and to any non-member for $1.25.—Editor

†Associate Engineer, Portland Cement Association, Chicago, Ill.

The benefits derived result from the ability to handle concrete of lower water content.

The following tabulation gives the maximum results obtained from the field tests of vibrators, in comparison with hand placing.

Water content 5.5 gal. per sack in each case

Placing Method	Cement Required Sacks per Yd.	% Sand in Aggregate
Hand Puddling	6.04	39
One Vibrator	5.10	31
Two Vibrators	4.80	31

The most important of the other observations were made in Missouri and later in Illinois during some tests on a vibratory concrete pavement finisher. These observations showed that improvements in quality or reduction in cement content similar in magnitude to those found in the Davenport tests could be effected.

It seemed that in the paving field especially it would be possible by vibration, if not in its present state of development surely in the more perfected stages to be expected in the future, to handle mixtures entirely outside of what is generally considered to be the workable range—mixtures such as have not been used since the days of dry-tamped concrete.

It was obvious that many of the problems of vibration would have to be solved in the field. The province of the laboratory seemed to be that of determining to what extent vibrated concrete complies with or departs from the general relationships that hold for plastic mixes. It is with the results of laboratory studies along this line that the ensuing discussion will be concerned.

Relation of Strength to Paste Quality. In this discussion, the term "paste" is used to designate the cement and water mixture which is the binding medium in the concrete. The quality of the paste is expressed in terms of the concentration of cement in the paste. This can be done in terms of the cement-voids ratio, in which case air bubbles as well as water are considered to be a diluent of the paste. The cement-voids ratio is that of Talbot and Richart[1] and is the same as the cement-water ratio used by Professor Lyse[2] except that the absolute volume units are employed and the space occupied by air is included.

These tests showed that, just as for hand-placed concrete, the quality of the cement paste is the chief factor determining strength, in both compression and flexure.

[1]Bulletin 137, University of Illinois Engineering Experiment Station, 1923.

[2]Cement-Water Ratio by Weight Proposed for Designing Concrete Mixes, by Inge Lyse; *Eng. News-Rec.*, V. 107, p. 723, Nov. 5, 1931.

Other variables were found to have effect, however, the most important being consistency of the mix and duration of vibration. Also (at a given value of cement-voids ratio), mortars were found to have higher strength than concretes, especially in rich mixes (high values of c/v) when tested in flexure. Prolonged vibration increased the strength of dry mixes slightly and reduced the strength of wet mixes, the greatest reduction being about 10 per cent in compressive strength. These variables are of minor practical importance, but of some theoretical significance.

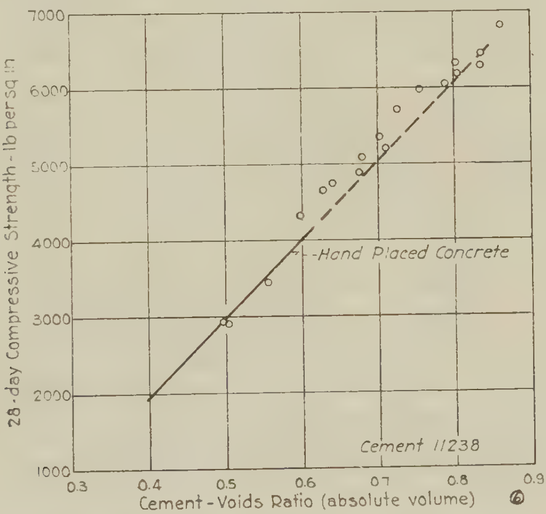

FIG. 6—RELATION BETWEEN COMPRESSIVE STRENGTH AND CEMENT-VOIDS RATIO FOR NORMALLY VIBRATED CONCRETE

Fig. 6[3] shows the relationships between compressive strength and cement-voids ratio for normally vibrated concrete. Fig. 9 shows the same relationship for flexural strength. It may be noted that the strengths of the top and bottom fibers in the vibrated beams are, in general, more nearly equal than in the hand-placed beams.

Freezing and Thawing Tests. At the beginning of this study there was some doubt as to the durability of concrete in which the coarse aggregate is highly concentrated. A number of the 6 by 12-in. cylinders were sawed longitudinally and the halves subjected to freezing and thawing while immersed in water. Cement contents of 4, 5, and 6 sacks per cu. yd. with various aggregate gradings and values of cement-

[3]Figure numbers are those employed in the complete text on which this report is based. See foot note on first page of this paper.

voids ratio were included. At present the specimens have withstood 110 cycles without any sign of trouble in the vibrated concrete. This is sufficient to indicate no impairment in resistance to weathering, due to vibration. Concrete of ordinary quality that has been exposed for 20 years to the climate of Northern United States without any signs of weathering has shown heavy scaling and disintegration under 20 to 40 cycles in this same freezing and thawing test.

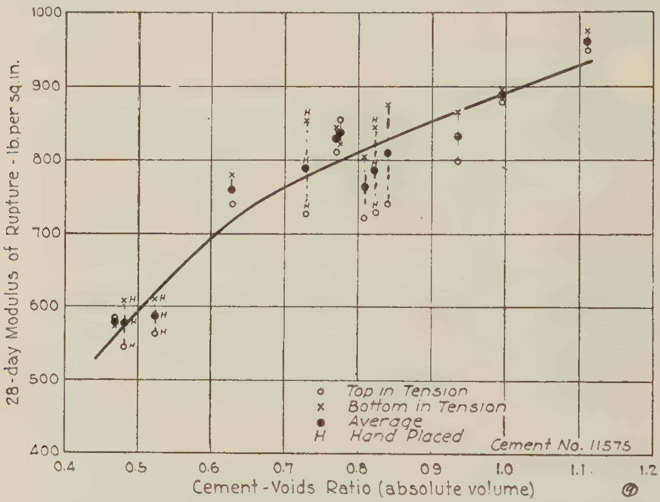

FIG. 9—RELATION BETWEEN MODULUS OF RUPTURE AND CEMENT-VOIDS RATIO FOR VIBRATED AND HAND PLACED CONCRETE

POSSIBILITIES OF VIBRATION

Having established that the paste quality, as indicated by the cement-voids ratio (or water-cement ratio*), is a sufficiently accurate index to concrete quality for all practical purposes, there will be considered next the magnitude of improvements in quality or reduction in cement content made possible by vibration.

At the outset it should be understood that the data to be presented represent laboratory results using a more effective vibration than the writer has seen in the field. However, they indicate the intrinsic value of this method of placing.

Use of Dry, Harsh Mixes. The full benefit of vibration, either on basis of strength or economy, is obtained by the use of dry mixes having a minimum of sand. This may be seen in Fig. 12, where the relation

*The cement-voids ratio is fundamentally the same as the water-cement ratio. For laboratory work of this nature, it is preferable in that it is a more accurate index to strength than the water ratio, since it takes into account the diluting effect of air bubbles. It was found that a given volume of air in the paste has about the same effect on strength as an equal volume of water.

between strength and per cent of sand is shown for three cement contents. The three solid curves show the relationship for vibrated, and the broken curve for hand-placed concrete, the latter curve showing the usual falling-off in strength when the quantity of sand is reduced below a proper minimum.

Taking 36 per cent sand as the optimum for hand-placed concrete (optimum for the quantity of cement and the materials used) the improvement in strength due to the drier consistency used when vibrating a mixture of the same cement content can be estimated from the vertical distance to the corresponding point on the curve representing 5-sack vibrated concrete. This shows an improvement of 1250 p. s. i. due to the drier consistency alone. The curve for vibrated concrete then shows that another 1350 p. s. i. could be added by reducing the sand to 23 per cent. This latter increase in strength requires the very minimum of sand. At 5 sacks per cu. yd., the practical minimum is about 28 per cent (with these materials) which would show a gain of about 900 p. s. i. over the strength at 36 per cent sand.

It should be remembered, of course, that these are indirect effects not due primarily to the reduction in sand, but to the lower water requirements brought about by this change in gradation.

Limiting Sand Percentage. In Fig. 12, it may be noted that unlike that for hand-placed concrete, the curves for vibrated concrete show no tendency to drop at the extremely low percentages of sand. This brings up a fundamental difference between vibration and hand placing.

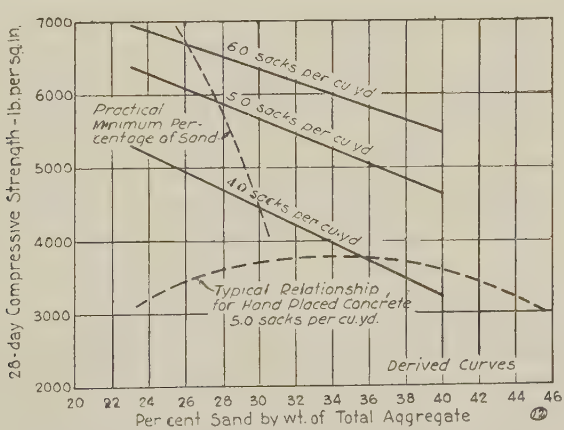

FIG. 12—RELATION BETWEEN COMPRESSIVE STRENGTH AND PERCENTAGE OF SAND FOR VIBRATED CONCRETE

Just as for hand placing, there is a lower limit of sand percentage for vibrated concrete, *but this limit is not indicated by strength results.* Under the conditions of these tests the behavior of undersanded mixes is quite different from what one might expect from experience with hand placing undersanded mixes in small specimens. When placing such mixes by hand the result is a mass containing irregular areas of honeycomb here and there over the surface and through the interior of the specimen, usually resulting in reduced strength. With vibration, however, compaction begins near the source of vibration where a plastic mass soon develops. As vibration proceeds, the volume of the plastic mass extends by incorporating more loose material. This continues untill all of the mixture has become plastic, or, if the gradation is outside of the placeable limit, until no more coarse material will be incorporated. *Any excess of coarse material will be left lying loose on top of the mass.* Thus, reduction in the percentage of sand below the placeable limit is manifested by the inability of the mix to absorb all of the coarse material. The underlying mass is homogeneous, and, with materials used in these tests, at least, it has normal strength.

In the paper, "Studies of Workability of Concrete"[4] it was pointed out that the minimum requirement for proper workability was that the mixes must have some degree of plasticity, which in turn required that they be of such nature that the solid particles remain in suspension during the placing process. It appears also that *in order that a mixture may be placed successfully by vibration, it must be plastic or become plastic under vibration.*

It should not be concluded that honeycomb cannot occur when vibrators are being used. In the field, vibration may not be sufficiently intense to be effective, or segregation prior to vibrating may make a part of the mix unplaceable. Furthermore, mortar may escape from leaky forms to the extent of rendering a properly designed mix unplaceable—vibration requires tight forms for this reason. Nevertheless, when vibration is successfully applied, it must bring the mix to the plastic state described above, and wherever that state occurs honeycomb cannot exist.

Cement Required for a Given Cement-Voids Ratio and Strength Developed at a Given Cement Content. With the grading and consistency properly adjusted, there is a large difference between the cement required to produce concrete of a given paste quality (cement-voids ratio) for the two methods of placing. This is shown in Fig. 13 where

[4]JOURNAL, American Concrete Institute, February 1932, *Proceedings*, Vol. 28, p. 419.

the relation between cement content and the cement-voids ratio for both hand-placed and vibrated concrete is shown.

Using the cement-voids ratio curves of Fig. 6 and 9, and the curves of Fig. 13, the relations between 28-day compressive and flexural strengths and cement content have been derived. These curves, which are shown in Fig. 14 and 15, are confined to the practical concrete range—c/v = .4 to .8 (9 to 4½ gal. per sack).

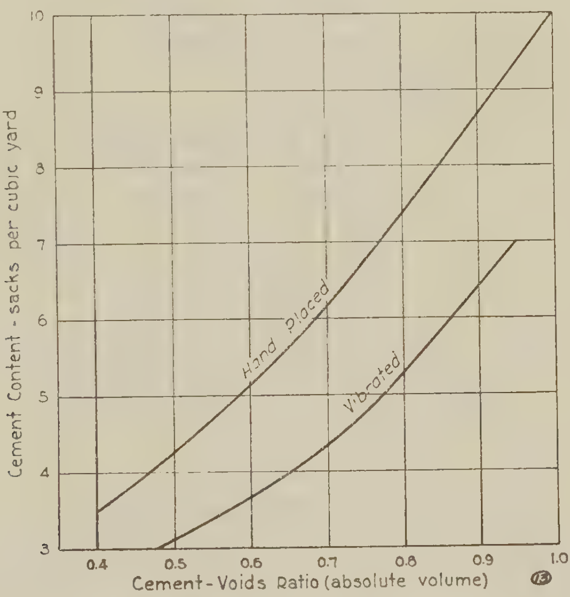

FIG. 13—RELATION BETWEEN CEMENT CONTENT AND CEMENT-VOIDS RATIO FOR VIBRATED AND HAND PLACED CONCRETE

HAND PLACED MIXES ARE OF GOOD WORKABILITY AT ABOUT 2-IN. SLUMP. VIBRATED MIXES ARE JUST UNDER THE LIMIT OF PLACEABILITY FOR LABORATORY VIBRATION

Fig. 14 and 15 may be taken as representative of the advantages to be derived from vibration as used in these tests. At a given cement content the compressive strength may be increased about 1750 p. s. i. and the modulus of rupture about 80 to 180 p. s. i. in the range of practical mixes. These charts show also that a given compressive or flexural strength may be produced with 1½ to 2 sacks per cu. yd. less cement than is required for mixes suitable for hand placing.

Significance. While the meaning of the above figures in terms of strength and cement content is easily comprehended, an equally significant conception of the value of vibration is had when the water

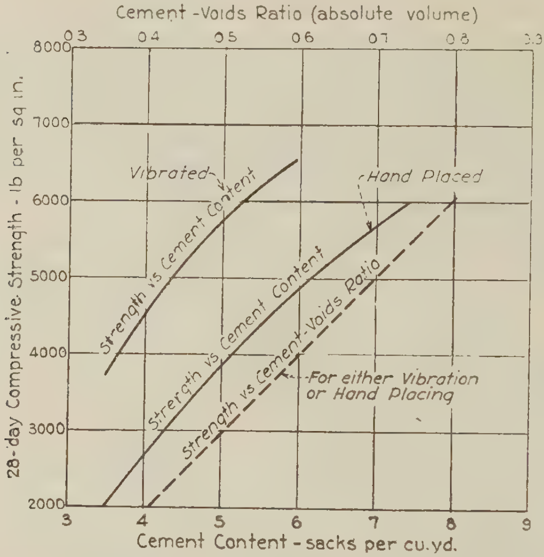

FIG. 14—RELATION BETWEEN COMPRESSIVE STRENGTH AND CEMENT CONTENT BASED ON FIG. 6 AND 13

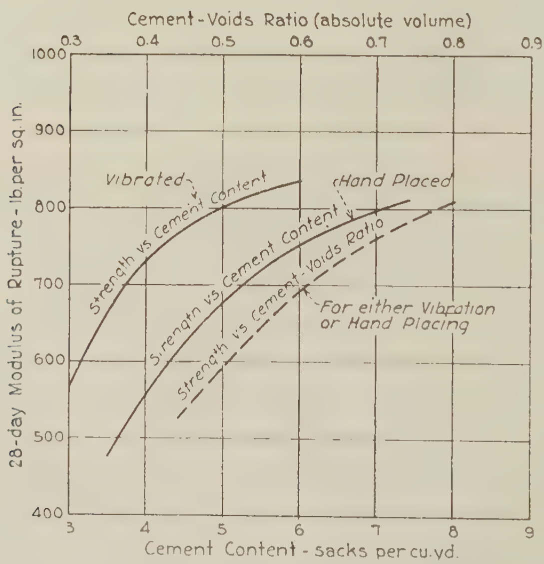

FIG. 15—RELATION BETWEEN MODULUS OF RUPTURE AND CEMENT CONTENT—DERIVED FROM FIG. 9 AND 13. BASED ON AVERAGE STRENGTH OF TOP AND BOTTOM FIBRES OF BEAMS

content of the mix is considered. Vibration makes it possible to reduce the water content of concrete 8 or 9 gal. per cu. yd., *thus eliminating more than a cubic foot of the least desirable component from each cubic yard of concrete.* This reduction in water content makes possible not only the substantial economies or improvements discussed above, but also it must have important effects on shrinkage.

Test results on volume change are not yet available, but estimates on the basis of existing data indicate that shrinkage may be reduced nearly one-half that for comparable hand-placed concrete. Such a reduction in volume change opens the way for the use of extremely high grade concrete, impracticable in the past because of the high shrinkage of the very rich mixes required. For example, 4¼ gal. per sack concrete *containing only 6 sacks per cu. yd.* can be placed by vibration, whereas it would require at least 8 sacks for such a water ratio when placing by hand.

While these laboratory studies have generally emphasized the possibilities of vibration in increased economy or greater strength through changes in mixture composition, the usefulness of vibration in placing under adverse conditions must not be overlooked. In places where it is almost impossible to get good results by hand placing, vibration, effectively applied, will cause the concrete to flow readily through small openings and into otherwise inaccessible places.

Readers interested in placing concrete by means of vibration are referred to other papers in this JOURNAL *and to contributions received at the 29th Annual Convention, publication of which is necessarily deferred to October 1933 (Proceedings, Vol. 30). A closure date for discussion will be announced at that time. Discussion of papers published here in part only (as indicated by footnote) will be acceptable from those who obtain the complete papers from the Institute. Contributors to discussion should in every instance send two copies of their discussions—one for the author of the paper discussed.*—EDITOR

The Use of Vibration in the Manufacture of Concrete Products*

BY MILES N. CLAIR†

Present day conditions require that every effort be made to reduce costs of production while maintaining or improving quality. Vibration, properly used, will help to attain this end in the production of concrete products by facilitating placing, by allowing the use of leaner mixtures and by reducing air voids. The use of vibration for placing and compacting concrete has increased rapidly during the last five years, but available engineering literature contains less than a hundred references to the subject of the use of vibration in relation to the production of concrete and, of these references, but two or three describe methods in detail and none gives fundamental data or theory. Published data on the use of vibration in the concrete products field are even less complete than for job made concrete. This lack of data is particularly serious as vibration improperly applied has been found to damage rather than improve concrete. The purpose of this brief paper is to discuss the variables envolved and to give some conclusions from investigations carried on under the direction of the author in the last decade, in so far as they have to do with the application of vibration to the actual manufacture of concrete products and thus to promote discussion and development of fundamental data. Many of the points mentioned are obvious, but are included as they are not equally apparent to all who deal with vibration. Likewise, the conclusions reached may differ from those with other experiences. The term concrete products as here used includes three types of plant made units; namely, concrete blocks, cast stone and precast reinforced slabs.

There are a surprisingly large number of variables to be considered when any study of vibration of concrete is undertaken and these may be conveniently grouped in relation to (1) characteristics of the vibration, (2) method of application, (3) characteristics of the mass to be

*Presented at the 29th Annual Convention, Chicago, Feb. 21-23, 1933.
†Vice President, The Thompson & Lichtner Co., Inc. Boston

vibrated and (4) purpose of the vibration. The characteristics of the vibration may include amplitude, rate or frequency, balanced or unbalanced, time of application, line of propagation, and violence of blow. The method of application may include application to concrete direct, through forms, through a vibrating table, direction of application, point of application relative to the top of the concrete, and period of application relative to placing the concrete. The characteristics of the vibrated mass include consideration of the kind and grading of materials, the proportions, the presence of reinforcement, the workability, the mass, and the dimensions of the mass being vibrated. The purpose of the vibration will vary and may include improvement of strength, of density, of surfaces, of speed of placing and economy of production.

If a container holding a material such as sand and gravel is struck a blow the particles are given an acceleration directly proportional to the force applied to the particle and inversely proportional to its mass. As the mass of each different particle may be different and the force transferred is different, the accelerations will be different and the displacement of the particles different. The large particles will move least and the particles close to the point of application the most. This may result in small particles moving away from the point of application of the blow and of small particles falling into air spaces between larger particles with resulting densification or compaction. Unbalanced vibration in one direction such as a hammer repeatedly striking the side of a container would tend to produce segregation, according to the above analysis, as the principal force would be in one direction. Balanced vibration, which would correspond to an alternate blow and jerk, would lessen this tendency but not entirely overcome it because the force of the blow would be most severe adjacent to the point of application.

Assume that the container is struck one blow every minute, it is apparent that particles come to rest between blows. Blows may attain a frequency at which particles continually in motion approach a condition of flowing. This frequency would be critical, giving a maximum of motion while a higher frequency would be objectionable, from the standpoint of efficiency, because the particle could not come to rest between blows and part of the energy of the blow would be wasted. The larger the particles, the lower the critical frequency.

Consider again the container with its mixture of sand and gravel, and this time a heavier blow is applied with the hammer. The force will be greater acting on the particles and the displacement greater.

The amplitude of the vibration of the particles from repeated heavier blows will be greater than for the previous case and, if the amplitude were increased sufficiently, the mixture not only would not be compacted, but actually would be made less dense because of the large displacement of the particles. There is thus, also, a critical amplitude beyond which there is displacement rather than densification. If we consider amplitude with frequency, it becomes apparent that a high amplitude requires a lower frequency than a low amplitude vibration.

To arrive at the conclusions in the last two paragraphs, consider a particle being vibrated by blows applied in a vertical direction. Under the action of the blow the particle moves a distance "a" vertically and drops back again under the action of gravity. the time required for the particle to fall distance "a" is given by the relation $t = \sqrt{\frac{2a.}{g}}$ It can be readily computed therefore that the following vibrations or blows per minute are critical for the amplitude given—that is the particle will receive the next blow the instant it comes to rest.

Amplitude	Blows per minute
0.5 in.	590
0.25 in.	835
0.1 in.	1320
0.01 in.	4160

Observation of the applications of vibration to concrete shows that the actual movement of the particle in the mass under proper vibration is somewhere between 0.01 in. and 0.1 in. The larger the particles the larger the displacement necessary and allowable. The amplitude of the vibration would be greater than that of the particle depending on the type of form, method of attachment of the vibration and the force of blow or size of the vibration.

The force of the blow will influence the acceleration and the amplitude of vibration in the material. An excessive blow will produce excessive displacement and possibly crushing of the aggregate and too weak a blow insufficient displacement. There is thus also a critical force of the blow.

The time of application of a given vibration will have an effect. One blow of the hammer on the container will probably not give maximum compaction. As the blows are continued, the particles will first tend to rearrange themselves into the voids and then proceed to segregate the sizes. Harsh, large size particles will require a longer period than others for the same size vibrator

The discussion has considered a dry granular material. Concrete may approach on one extreme in its characteristics the sand and gravel

we have been considering, and, on the other extreme, a fluid. The more workable or fluid the mixture, naturally the less the force required to cause motion and the less the work required to transport and consolidate the mass. This indicates lower frequency, lower amplitude, and shorter time of vibration, or what might be termed less severe vibration, the more workable the concrete. As an example it was found that zero inch slump concrete will require an expenditure of 20 ft-lb. of energy per minute for two minutes to compact it, while two inch slump concrete will require only 10 ft-lb. of energy per minute for one minute. Definite practical values from practice for period, amplitude, and frequency will be given later in this paper.

The method of applying the vibration will determine, to a large extent, its effectiveness. In the practical application generally, the type of vibration is limited because of the relatively limited range of characteristics of the commercial vibrators available. The problem becomes one of application of the vibration so as to obtain the best results. Vibration may be applied to (1) the forms, (2) the surface of the concrete, (3) the interior of the concrete mass, and (4) combinations of these. The application to the forms is the most common in concrete products manufacture and may include rigid attachment or clamping to the form, loose attachment or hanging on to the form and manually applying against the form. Rigid attachment requires time for attachment and removal and is not generally used except on vibrating tables or block machines. Holding the vibrator to the form does not produce uniform results, as the force of blow will vary and should be used only as a temporary expedient. The more rigid the form the heavier the blow to produce a given amplitude of vibration in the concrete. A light form is, therefore, desirable, but the construction must be air tight so that opening and closing of a joint under vibration will not pump air into the concrete. A form that is too flexible will not transfer the vibration evenly across the section. Form vibration is generally excellent for slumps from two to four inches.

Application of the vibration to the surface of the concrete through a small shoe or plate is desirable for harsh, lean and dry mistures, but does not permit uniformity of control and may interfere with the reinforcement unless rigidly held. The method is most used where the volume of concrete is large and a very dry mixture is required. Block machines as generally made have tamping rather than vibrating action. The method does not apply where the slump is more than one inch, as excessive water will be drawn to the surface at the point of application.

The internal vibration of concrete by inserting a vibrating arm into the body of the concrete would appear to be most efficient in removing

air because it provides an easy path along the arm for the escape of the air. Higher frequencies and lower amplitudes may be used because of the direct action of the vibration on the concrete. There is necessary, however, uniformity of control if the mass is to be uniformly vibrated. The method is equivalent to direct form vibration where the depth of concrete is small, as the spade then rests on the forms. The method is applied to medium consistencies (2 in. to 4 in. slump) and deep sections.

Combinations of the various methods of application may be used to advantage. Surface vibration has been supplemented by form vibration where dry mixtures and good surfaces are required.

Vibration appears to allow a film of cement and water to form on the face of the mold where the vibration is applied. If the vibration is not properly controlled, under this film will be found numerous voids formed by bubbles of air.

The effect of application of vibration depends on the direction. A vibrating table may produce a horizontal, vertical or a combination vibration. Form vibration is usually horizontal. Surface vibration is generally vertical. Internal vibration is vertical or horizontal, or a combination of the two. There appears to be little practical difference between the results obtained by either vertical or horizontal vibration. Vertical vibration may produce segregation vertically, while horizontal vibration may produce segregation horizontally. The combination vibration would appear to be the best, but envolves more expensive equipment. The surface next to the bed plate in vertical vibration will generally have the best finish. The face opposite the vibration in horizontal vibration will generally have the best finish.

The vibration should be applied in such a position that the whole section will have about the same amount of work done on it by vibration. A vibrator applied to the top of each side of the form and operating during the filling of the form will approximate this result if the form is not too deep, because the bottom receives less of the force of vibration and is vibrated for the longest period. A vibrator applied to the bottom of a form, as is done on the usual vibrating table, tends to over vibrate the bottom concrete, if the section is deep—compared to its thickness. Surface and internal vibration must be applied to cover the entire area evenly.

Vibration, as discussed elsewhere under time or period of vibration must not continue too long otherwise it will tend to segregate the

concrete. One minute or less is the usual limit. Vibration is best started as the concrete placement is started and stopped a few seconds thereafter. The placement of the concrete may require a longer time than is permissible for the vibration. In this case the vibration time should be distributed over the placing period or a smaller vibrator used and moved up with the concrete in the forms. Vibration obviously should not be applied after concrete has started to harden.

It has become apparent, in the discussion so far, that the characteristics of the mass to be vibrated largely determine the kind of vibration necessary. A large mass obviously requires a heavier vibration than small mass. A ¾ in. unbalanced type vibrator, 75 lb. air pressure, gave good vibration hung direct to steel forms made of ¼ in. steel plates, containing ⅔ cu. ft. of concrete. A 1½ in. vibrator of the same type produced an excessive vibration when so applied, but when applied to a similar form containing two cu. ft. of concrete it gave good results. A given mass of concrete in a thin slab is vibrated more effectively from the bottom than the side. The same mass as a unit of 4 in. width and 2 ft. height is more effectively vibrated from the side, or internally.

Concrete made with well graded materials requires less vibration to compact it. Gravel is more quickly compacted than crushed stone with the same size vibrator. Cinders, slag and burnt clay or shale require a more rapid vibration and a longer period but the results are more marked as mixtures can be placed by vibration that cannot be worked by any other procedure. The rich mixtures are well lubricated and require very short vibration. The more workable the mass or the higher the slump, the less the vibration necessary, that is, less time, less amplitude and less speed or frequency. Segregation and rapid water gain result from vibration of concrete with slumps over 4 in. and it is doubtful whether vibration has any value for the higher slump concrete, except to expedite placement and then it must be used with caution. Reinforcement must be securely fastened or it will rise from prolonged vibration.

Vibration is a means to an end and that end is not the same in all cases. In the manufacture of concrete products, vibration is most frequently applied to improve strength and density of the product. If the proportions are held constant, vibration may improve strength by reducing the air voids, thus increase the compactness of the mass and by separating out some of the water reduce the water-cement ratio. There are many tests that show no gain in stregnth from vibration and others that show as much as 100 percent increase. If the mix

is workable and there is plenty of paste present, the reduction in voids and strength change by vibration will be small. Density is increased by the removal of air and excess water which thus reduces the porosity of the unit. A dense surface, free from bubble holes is most desirable for cast stone that is to be polished. Vibration of excessive amplitude that pumps air into the concrete will give surfaces badly pitted. Mixtures too dry or too wet and the use of some types of oil on the molds will increase the tendency towards the formation of holes.

Vibration is also employed as an aid to placing material for concrete products. It gives remarkable aid where dry mixtures must be placed in thin slabs of 12 to 20 sq. ft. area. The effect of vibration is similar to that taken advantage of in its application in the field of conveying, where materials have to be moved by vibration methods at the rate of 50 ft. per minute. A large amplitude (½ in.) and moderate frequency, 2000 vibrations per minute, are desirable for placement purposes.

In the practical case the mass of concrete and its shape are known. The choice is between electric and air powered tools. Electrical tools developed to date are less flexible in regard to modification of their characteristics and more costly. Air powered tools are low of cost but require expensive auxiliary equipment. Air equipment, however, is generally available at concrete products plants. If the product is light enough to be handled by one man a vibrating table will probably provide maximum ease in production. The forms, consisting of bottom and sides, can be placed on this table, filled during vibration, the top levelled off with a strike bar and trowel and the unit removed and stored for stripping. Units 12 x 24 x 1 in. of approximately 1:3 mixture, cement and granite chips, of 2 in. slump, were successfully vibrated on a table 30 x 42 in. in plan. The table was made of ½ in. steel plate, supported on heavy springs with a 1½ in. unbalanced pneumatic vibrator hung from the center. The vibrator operated on 80 lb. pressure with a frequency of 3000 vibrations per minute. The vibration period was 50 seconds. The molds were made of wood sides and plywood bottoms. The procedure adopted (after an investigation varying the size of vibrator, legnth of vibration, time of application of vibration and consistency of mixture) included starting vibration with start of placing concrete, stop during screeding to level and vibrate for two seconds to level off after screeding. The same unit has been made as successfully on a table 48 x 78 in. with a ¼ in. steel plate reinforced with angles to the center of which was attached a 3 in. pneumatic vibrator.

Units 4 in. thick, 12 in. high and 24 in. long were made with two finished faces. The casting was done in steel molds made of channels

and ¼ in. plates, 4 specimens to a mold. An investigation including variation of time of vibration relative to placing of concrete and point of application showed that the procedure that gave the most dense concrete, best surfaces, and highest strength included a 1:3 mixture of cement and granite chips, 4 in. slump, vibration during placing with a ¾ in. pneumatic unbalanced type vibrator, attached to a yoke resting loosely on the top of the form just at the stop between the units, and a time of vibration of one minute.

Precast reinforced slabs 2 by 6 ft. and 1½ in. thick were made with a mixture approximately 1:4 cement and cinder aggregate. The forms were of ⅜ in. thick angles and No. 8 gauge steel pans. The consistency of the concrete was one inch slump. The best form of vibration was found to be two pneumatic vibrators, unbalanced type, 75 lb. pressure, 1½ in. size, hung in a horizontal position on the angles during placement of the concrete, which required about 1½ minutes. Levelling off was done during vibration.

It must be apparent from these illustrations and the discussion that until more fundamental data are obtained, vibration practice in the concrete products field must be determined by previous experience plus thorough investigation for the case considered. What is the best will be determined by cost and by the characteristics of the product measured by tests for absorption, compressive strength, modulus of rupture and voids.

Readers interested in placing concrete by means of vibration are referred to other papers in this JOURNAL *and to contributions received at the 29th Annual Convention, publication of which is necessarily deferred to October 1933 (Proceedings, Vol. 30). A closure date for discussion will be announced at that time. Contributors to discussion should in every instance send two copies of their discussions—one for the author of the paper discussed.*—EDITOR.

Vibratory Finishing Machine For Concrete Pavements*

BY F. V. REAGEL†

To explore possibilities for reducing the cost of constructing concrete pavements, the Missouri Highway Department sought the answer to two major questions: (1) How does vibratory finishing affect the quality of the concrete? (2) What economy if any does it make possible?

The investigation consisted principally of making a comparison between two types of pavement finishing machines, one a vibratory machine and the other a regular-type Ord finisher. The tests were conducted last summer on a Federal Aid project on Route U.S. 50 in Moniteau County.

The vibratory finishing machine used was a new, specially designed "Lakewood," and was equipped with five "Jackson" electric-motor vibrators. Its screeds were considerably wider and heavier than usual. Three of the vibrators were mounted on the front screed and two on the rear screed. This machine was furnished to the Highway Department for these tests through the cooperation of the Lakewood Engineering Co., and the Electric Tamper and Equipment Co. This same machine has since been used in making similar tests in Illinois, and is to be tried in Michigan.

The pavement on which the Missouri tests were made was the American Association Standard design. It was a 9-7-9 slab, 20 ft. wide having lip curbing and bar-mat reinforcing. The aggregates were crushed Burlington limestone and Missouri river sand, taken from stockpiles and proportioned on the basis of dry rodded volumes. Both aggregates were measured by weight with allowances for free water present.

In the regular procedure on this job, a 1:2:3.58 mix was used; concrete was placed at a consistency equivalent to about a 2-in. slump; finished with a standard Ord finishing machine equipped with an auxiliary tamping bar; a flexplane machine was used for installing the longitudinal center joint and lip curbing was placed by hand labor.

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The pavement was covered for twenty-four hours with wet burlap, and then treated with flake calcium chloride.

For the duration of the experimental construction, the vibratory machine was placed on the forms and made to travel along in front of the standard finisher. However, the operation of the vibrators was discontinued at intervals to revert to standard finishing and construct test sections as standard of comparison.

Preliminary tests were made in which the vibratory machine was used both on the regular-mix concrete and on other, arbitrarily selected, mixtures to show the possible working range in consistency and mix. These observations at once indicated that the vibration method was an effective means of handling dry mixtures, that is, mixtures having a slump of about one half-inch. In fact, it was found that dry consistency was quite essential to the satisfactory operation of this machine. The standard mix as well as some of the other mixes tried contained such excess of mortar, particularly when placed at a consistency suitable for ordinary finishing, that the action of the vibrators worked a great deal of mortar to the surface of the slab, and before the final floating and straight-edging could be satisfactorily completed, it had to be pushed off over the side forms. An attempt to use these same mixtures at drier consistency proved futile. In the standard mix, a reduction in mixing water of $\frac{3}{4}$ gal. per sack of cement, while reducing the slump from 2 in. to about $\frac{1}{2}$ in., did not prevent an excess of mortar from being brought to the surface under the vibrators. Obviously, for vibratory finishing, this mixture (with 39 per cent sand, absolute volume) was considerably over-sanded.

Guided by these preliminary observations, the investigators designed two separate series of mixtures to be used and finished by the vibratory machine. The first series consisted of four mixtures, each having a different ratio of sand to coarse aggregate, but for concrete having a cement factor equal to that of the standard mix, or 1.44 bbl. cement per cu. yd. of concrete. The purpose of these mixtures was to determine what ratio of sand to coarse aggregate (for the particular aggregates being used) was best adapted to vibratory finishing and at the same time to make comparison possible with standard finishing on the basis of effects on quality of concrete produced in mixtures approximately equal in cost. One of these special mixtures, the one in which the aggregate was 32 per cent sand, was selected as the best, that is, superior to the others from the standpoint of workability under the conditions necessary to the operation of the vibratory machine.

A second series of mixtures was designed to determine the practical limit to which the cement content of the vibrated concrete could be reduced, and to compare a relatively lean vibrated concrete with the standard non-vibrated concrete.

This series also consisted of four mixtures, but included the 32 per cent sand mixture of the first series. The other three mixtures were designed for lower cement factors. For these four mixtures the cement factors were respectively 1.44, 1.30, 1.20 and 1.13. All gaged to the same consistency, ½ to 1-in. slump, and used alternately in the construction of test sections 100 ft. long.

The greatest difficulties encountered, viewed from the standpoint of construction, were those arising from the attempt to adapt the ordinary equipment and methods, not connected with the operation of the vibratory finisher, to the handling of concrete which was, according to general understanding, unworkable. The dry consistency of the concrete, combined with that feature of being undersanded, hindered the progress of the paver and also required the use of additional hand labor to spread the concrete in front of the finishing machine. The vibratory machine apparently had the capacity for finishing concrete of slightly drier consistency than that which could be moved and distributed properly over the subgrade by any reasonable amount of hand labor.

Some of the mixtures, especially those containing the smaller percentages of sand, segregated as they were being discharged upon the subgrade. The most noticeable feature was the separation and apparent isolation of particles of coarse aggregate along the edges of each batch. Subsequent core tests showed some honeycomb in the finished pavement, thus proving that vibratory finishing cannot be expected to offset the lack of proper spreading.

The extraordinarily dry mixtures of concrete placed with the vibratory finisher made difficult work for the hand laborers in molding the lip-curb and in operating the flexplane machine to install mastic in the center groove. The floating and straight-edging, however, were not appreciably affected except in that the workmen were required to follow more closely behind the finishing machine.

The surface of the slab after being finished with the vibrator appeared to be in excellent condition. Only a very thin layer of mortar was on top, and practically no free water. This made possible the almost immediate covering with wet burlap for curing.

The concrete in sections of pavement finished by the vibratory machine seemed harder and appeared to set quicker than the non-

vibrated concrete. Sooner than concrete would normally be expected to begin to harden, these vibrated sections could be walked upon without any appreciable movement of the pieces of coarse aggregate. This can be attributed to the dry consistency and low mortar content; and it may, though not necessarily, indicate more nearly perfect compaction resulting from vibration.

Observations of the various test sections made in the morning following the day of placing failed to reveal any noticeable differences in surface characteristics. When the forms were removed the edges of the slab were examined to detect differences in the extent of honeycombing. For that part of the surface along the edges below the lip curbing, there was no more honeycombing in test sections of one mix and finishing method than in any other sections on which a different mix or finishing method had been used. However, the edges of the lip curbing on the vibrated sections were considerably worse honeycombed than those on the unvibrated sections. This undoubtedly was due to the harshness of the mixtures used and the difficulty in building the lips by hand.

The test for the quality of the concrete consisted of drilling five 6-in. cores from each of the various test sections, rating them visually, and testing them for density and strength.

All of the cores were examined visually by two observers who worked separately giving each specimen a rating number. The ratings were based on the apparent segregation, porosity of structure, and honeycombing in the concrete. The classification was made without regard for mix or finishing method but later the data were grouped to ascertain what correlation there was. The results indicated that the vibrated concrete was, in general, of a slightly better quality than the non-vibrated.

The density tests were made on cores taken from the standard test sections and on cores representing three of the vibrated mixtures. The measure of density used in comparing these specimens was the per cent of voids in the concrete. The voids were calculated from the data derived by determining the apparent specific gravity of the cores, and the true specific gravity, or specific gravity of the pulverized material. The average densities thus obtained for the various groups of cores were found to differ by such small amounts that it was felt no strict, quantitative comparisons were warranted. No doubt the discrepancies in these tests were sufficient to overshadow the variations observed.

In making the strength tests, all of the cores were tested in compression at the age of 28 days. Cores of 6 in. diameter varied in length from 7½ to 9½ in. after being capped. The strengths as measured were corrected to take into account the effect of variation in height-diameter ratio.

The results show that all vibrated mixtures designed for a cement factor equal to that of the standard mix gave higher strengths than the standard. The difference based on average strengths was 580 p. s. i. or 11.8 per cent in favor of the vibrated concrete. This is considerable in view of the fact that the mixtures compared yielded exactly the same volume of concrete per unit quantity of cement.

Cores which represented the vibrated mixture containing 1.30 bbl. of cement per cu. yd., showed on the average 400 lb. greater strength than the standard; and the cores representing the vibrated mixture which had a 1.20 cement factor gave an average strength only 200 lb. below that of the standard. This may be summed up by saying that in the former case a ten per cent reduction in cement was accompanied by a nine per cent increase in strength, and in the latter, a seventeen per cent reduction in cement was accompanied by less than five per cent reduction in strength.

The other vibrated mixture which was the one having a cement factor of 1.13 was the leanest mixture that was used, and it was not repeated in a sufficient number of test sections and used under proper conditions for making important comparisons. However, the minimum strength was above 3500 p. s. i., thus placing it in a range of acceptable quality for pavement concrete insofar as strength is concerned.

Regardless of mix proportions and method of finishing, the average strengths for the various test sections, plotted against water-cement ratio, gave an average curve of characteristic trend. This was interpreted to show that the increase in strength accorded to the vibrated concrete was really a result of decreased water cement ratio. This does not lessen the advantages of vibration because without the vibrators such low water cement ratios could not have been employed.

All of the vibrated concrete, containing not more than 5½ gal. of water per sack of cement, gave strengths equal to, or greater than, the average strength of the regular-mix concrete which on the average had a little less than 5½ gal. per sack of cement. The core tests also indicate that for any given water-cement ratio, the vibrated concrete had smaller variations in strength than the non-vibrated. This probably means that the vibrators gave a more uniform degree of

compaction than was obtained by means of the screeding and tamping, as done by the Ord machine.

A detailed condition survey, of the various test sections after the pavement had been subjected to traffic for four months, disclosed no differences in the appearance or serviceability between the vibratory-finished and the standard test sections. Although there were several small areas of thin laitance scale and some spots that had been pitted and scratched by the shouldering crew, none of these defects could be attributed to variations in the inherent quality of the concrete.

In conclusion it may be stated that the results of this experiment definitely indicated the possibility of reducing the unit cost of concrete in pavements without sacrifice in quality. In consideration of the fact that cement is the most expensive material normally used in making concrete, the discovery of a method whereby concrete of standard quality, or perhaps slightly improved quality, can be produced with a smaller amount of cement per unit volume of concrete certainly means a potential saving in the cost of materials.

Of course, under conditions prevailing during this experiment, the increased labor cost probably offset the saving in materials cost. Nevertheless, in view of the fact that the design of the vibratory equipment and the application of the method are still in the experimental stage it is probable that further investigations of the nature of the Missouri experiment will prove the possibility of considerable economy.

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The Temple of Light*

BY ALLEN B. MCDANIEL†

What is the Baha'i Temple? What is its significance? It is being erected as the first universal temple of the world-wide religious movement known as the Baha'i Cause which originated in the Holy Land, about seventy years ago, with the advent of remarkable characters who gave their teachings to the world. Abdul Baha (Servant of God), a great teacher came from the Holy Land westward, first through Egypt and then Europe and America, in 1912, with a universal message of the oneness of mankind and of peace. In June, 1912, he spoke of the universal temple and at that time went out to Wilmette, on the shores of Lake Michigan, where the north channel of the Chicago Drainage District takes the water out of the Lake, and laid a stone saying here would be erected this first universal temple of the Faith in the world; it would be the center of community life, surrounded by other buildings such as the educational institution, the hospital, the hospice to take care of visitors, the community home for orphans, the aged and indigent, and provide housing for the various other humanitarian agencies of the community. He said that this whole community would be known as the Mashriqu'l Adhkar--that is the Arabic for Dawning Place of the Mentionings of God.

About seven years later at an interesting convention in the Engineering Societies' Building in New York City, followers of the Faith from all over America with many visitors from other parts of the world assembled to select a temple design. Among six different sets of designs presented, was a very beautiful and unique plaster model submitted by one who at that time was relatively an unknown architect, Louis Jean Bourgeois. To aid in the selection the convention called in as consulting architect H. Van Buren Magonigle, who made this statement in regard to Bourgeois design:

> Mr. Bourgeois has conceived a Temple of Light in which structure, as usually understood, is to be concealed, visible support eliminated as far as possible, and the whole fabric to take on the airy substance of a dream; it is a lacy envelope, enshrining an

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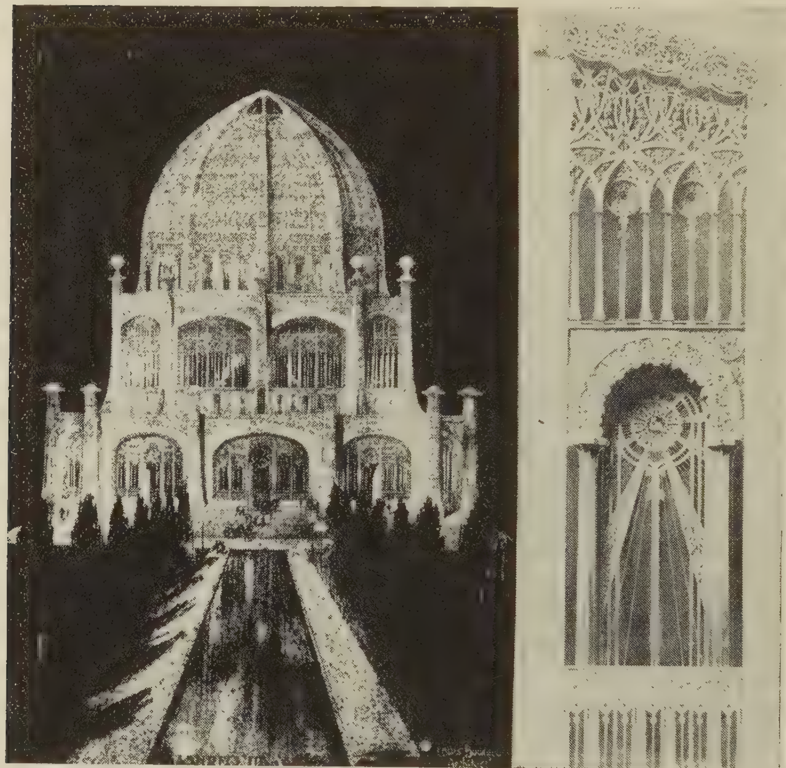

BAHA'I TEMPLE AS CONCEIVED BY ITS ARCHITECT—LOUIS BOURGEOIS
AT THE RIGHT—A DETAIL OF DELICATE TRACERY WHICH IS TO ENVELOP THE PRESENT PURELY STRUCTURAL EDIFICE OF CONCRETE, STEEL AND GLASS

idea; the idea of Light, a shelter of cobweb interposed between earth and sky, struck through and through with Light—Light which shall partly consume the forms and make of it a thing faery.

Later on Mr. Bourgeois spoke in these words concerning the significance of his design as expressed in this beautiful model:

The teachings of Baha'u'llah (Glory of God) unify the religions of the world into one universal religion, and as we know that all great historic religions developed a new architecture, so the Baha'i Temple is the plastic teachings of Baha'u'llah. In the Baha'i Temple is used a composite architecture, expressing the essence in line of each of the great architectural styles, harmonizing them into one whole.

Mr. Bourgeois' design was selected for the temple.

The late Major Henry J. Burt first structural engineer for the temple trustees, was the chairman of a board of engineers and architects selected to consult with the trustees who were given the task of pro-

viding the ways and means for the construction of this unique structure. It was my privilege to be a member of that board and it was very interesting to get Mr. Bourgeois' point of view and learn how this very remarkable and unique vision, as he called it, came into being. He stated that over a period of years he had been working on designs for the temple of Peace at the Hague, a seven-sided structure, crude and unsatisfactory. One night he had a vision and it was so strong that he got up and lighted the lamp in the little home where he lived then, in West Englewood, New Jersey, and made some sketches on the backs of some envelopes, of a nine-sided building with minarets at the corners. The only part of the vision that came to him then was just the two stories.

For three months I worked and placed all kinds of domes on those two stories and could not get anything that was satisfactory; nothing seemed to be in proportion. I became impatient and was almost frantic trying to complete this design. Then one morning I had about given up hope when, in a flash of light, I was awakened and saw the dome of this building. It was *on* the building. I got up and snatched a piece of wrapping paper and made a sketch of the building and the dome and then went back to bed. I arose the next morning and there I found my temple. I realized that this temple was so different and so new and unique, that any drawings I might prepare would not give a satisfactory idea of it. I decided that the thing to do was to make a plaster model.

Thirteen years ago that model was shipped through the country and exhibited in museums and art studios. It is interesting that the money for the site which was purchased just prior to the time that Abdul Baha came to this country and the building of the temple was decided on, was secured entirely by voluntary contributions from followers of the Faith throughout the world; all nations, all creeds, rich and poor, high and low, Gentile and Jew, Zoroaster and Buddhist, have contributed to this temple, largely because most of them are poor, at some sacrifice.

The building, as originally designed by Mr. Bourgeois, is nine-sided and all of the sides are alike, with a doorway at the center, flanked on either side by two ornamental windows and enclosed with a flat arch. At the intersection of the sides, there is a pylon or minaret. The sides are curved, concave and that, as Mr. Bourgeois explained to us, is a symbol of the building, extending outstretched arms. The first story is 36 ft. high, on a circular foundation with 19 steps from the ground surface up to the main floor; a second story 45 ft. high is considerably recessed, again with nine sides each having a group of windows. Above is the clerestory 19 ft. to the dome which rises little over 200 ft. above the ground, just about the same height as the Capitol dome in Washington. Unique are the temple's nine sides. The second

story is offset in reference to the first, a feature that projected considerable comment when the design was first exhibited, because everybody said that the ribs of the second story should be in line with the pylons of the first story. Mr. Bourgeois stated that he had introduced a new principle in design; the ribs are curved and abut against the arched faces of the building. The criticism at that time was so great that Mr. Bourgeois made a sketch swinging the whole building around through twenty degrees so that the ribs and the minarets of the first gallery were in line with the pylons of the first story and the whole design became lifeless and dead. The feature of the nine ribs which extend over the dome is their unique termination at the top; symbolizing hands lifted in prayer. Symbolic in the design are:*

*In the geometric forms of the ornamentation covering the columns and surrounding windows and doors of the temple, one deciphers the religious symbols of the world. Here are the swastika cross, the circle, the triangle, the double triangle or six pointed star (or Solomon's seal), but more than this the noble symbol of the spiritual Orb, the Greek Cross, the Roman or Christian Cross; and supreme above all, the wonderful nine pointed star, figured in the structure of the temple itself, and appearing again and again in its ornamentation, as significant of the Spiritual Glory of the world today.

The nine pointed star reappears in the formation of the windows and doors, which are all topped by this magnificent allegory of spiritual glory, from which extend gilded rays covering the lower surfaces, and illustrating, in this vivid and artistic limning, the descent of the Holy Spirit.

The numbers 9 and 19 recur again and again in the temple, illustrating its basic principle of Unity—nine being the number of perfection, containing in itself the completion of each perfect number cycle, and 19 representing the Union of God and man, as manifested in life, civilization and all things.

As a member of the Advisory Committee of Engineers and Architects making a study of this structure for the Temple Trustees, we had a very intersting time for a period of about twelve years. Many people, when they saw the model in the Engineering Society Building there in 1919, said, "That is very beautiful but it cannot be built. That lace-like design of the dome and of the windows and the whole thing is a very lovely conception, but absolutely impracticable".

So it became the function of this committee of engineers and architects, to try to materialize the vision of the architect. I think as we look at the design many of us, especially of a mathematical type of mind, see the unusual opportunity for working out an elastic structure. This idea was considered by the Board of Engineers and Architects some years ago, but after considerable discussion it was finally decided to consider this project in two different parts. First, to build a skeleton structure of the general form and shape of the design, and

*From *Architectural Record* June, 1920.

then to clothe that structure with what Mr. Magonigle has referred to as "a lacy envelope;" that is, clothe the skeleton or superstructure with ornamentation, and so the work has proceeded along those lines.

It started in 1921 with the building of the foundation. Over a period of seven years the people of this community, especially the North Shore passing up and down along Sheridan Road, have been wondering what it was all about, and when the work was to go ahead. The foundation work was finished in 1922, and then, on account of lack of funds, the work did not proceed until August 1930, when a contract was let to the George A. Fuller Co. for the superstructure. Actual work began early in September of that year and was completed in June, 1931. That is the structure as it is today and which many of you will see during the visit tomorrow morning. Since the work was virtually completed in June 1931, however, much has been done in the development of the mechanical plant and the utilities in and about the temple.

The most interesting piece of research in connection with the temple —and there have been several,—was in connection with the ornamentation. The Board of Engineers and Architects looked up all the literature in regard to temples or buildings of this general type and design, and visited many buildings in this country and some abroad, trying to learn what material was best adapted to the lace-like ornamentation. We considered natural stone, cast stone, architectural concrete, the various metals, such as steel, cast iron, and non-ferrous alloys, terra cotta; everything we could think of or could learn had been used in building construction was considered. After about eleven years of continuous study, economic as well as material, structural and engineering in character, we finally decided that, in view of the requirements of the architect to produce a structure that would be pure white in color and with a radiant surface, a "Temple of Light," a particular form of architectural concrete in which impervious and durable stone would be used in the surface, such as white opaque quartz and of a highly reflecting surface property or character, was the most suitable material. On June 6, 1932, a contract was entered into with the John J. Earley Studio, Washington, D. C., to proceed with the making of the ornamentation for the dome, and that work is now well under way at the Rosslyn, Va., plant of the Earley Studio. It is my understanding that opportunity will be given later on to tell the Institute about many of the various problems that we are meeting in the handling of the ornamentation, both its preliminary manufacture and erection at the temple.

The Project of Ornamenting the Baha'i Temple Dome*

BY JOHN J. EARLEY†

Twelve years ago, last August, two gentlemen came to my studio in Washington. They came unexpectedly and they brought with them only the photograph of a plaster model. They had been sent by a mutual friend, an engineer, deeply interested in the work deing done with concrete by this studio, who suggested that we might offer a solution for their problem. One of these gentlemen was Mr. Louis Bourgeois, an architect, and the most unusual personality I have met in that profession. The other was Mr. Ashton, his friend, and the pohotgraph which they brought was of a Temple, the most exotically beautiful building I have ever seen. It came up out of the earth like the sprout of some great plant bursting out to life and growth.

Mr. Bourgeois explained that he was the architect of the building and a member of the Baha'i Faith who believe themselves to be the children of a new era, who believe that they have received a new Manifestation. It soon became clear that this Temple was the dream of Mr. Bourgeois' life, that all his hopes and ambitions were centered in it, and that he believed himself to have been inspired to design a temple unlike any other in the world, so that it might be the symbol of a new religion in a new age. At that moment he was anxiously seeking a material with which to build it and someone with the ability to understand his work and the skill to execute it. He left with me the photograph, after autographing it. I have it still. It marks the beginning of the project for me.

In the time which intervened between this meeting and the death of Mr. Bourgeois about two years ago, there developed between us an interesting and instructive friendship. We studied this temple with all its ramifications of form, of treatment and of meaning as a preparation for the time when work on it would be begun. It was strange, in a way that we of the studio should have given so much thought to it. We had no authority to do so and as a matter of fact we were not commissioned to do the work until this summer just past. But somehow it

*Presented at 29th Annual Convention, Chicago, Feb. 21-24, 1933.
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always seemed to be our work. We understood it, we had the material and were equipped to do it. The architect was interesting to us and we to him. And then there was the job itself, a thing to fascinate the imagination. A temple of light with a great pierced dome through which by day the sunlight would stream to enlighten all within and throught which by night the Temple light shone out into a darkened world. When at night we look into the sky we see only the stars but could we see the orbits of the stars how wonderful it would be. Great curves intertwining in wierd perspective. Ovals, circles, and vesicas of endless variety twisted and woven into some great cosmic fabric. This is the theme of the dome, the courses of the stars woven into a fabric. But this is not all, inter-woven with the courses of the stars in the pattern of the dome are the tendrils of living things, leaves, and flowers, because no symbol of creation would be complete without a symbol of life. Lifted above the dome are nine great ribs, nine aspirations that mount higher than the courses of the stars. I wonder after all if it was strange that we of the studio should have given so much thought to this project?

The drawings left to us by the architect adequately illustrate his ideas about the decorations of the dome but they do not pretend to show a method for making the dome nor for attaching it to the steel skeleton. Among his drawings are some of the most extraordinary full sized details of ornament. There is one of a panel in the field of the dome which is seventy feet long, another of the face of the great rib which is ninety feet long. Each of these drawings was made in one piece in a loft building on LaSalle Street in Chicago where Mr. Bourgeois stretched out on the floor a great sheet of paper and with his pencil tied to the end of a long stick he drew in great sweeps, in a manner never to be forgotten, the interlacing ornament of the dome. One line through another, under and over, onward and upward until the motif was completed. Never have I seen a greater feat of draftsmanship nor a more interesting draftsman than Mr. Bourgeois. Most surprising of all perhaps is the approximation to accuracy which he maintained in these great drawings in spite of the disadvantages under which he worked. He was obliged to stand on the drawing which he was making and his only view of the whole was from the top of a step ladder. It became necessary after the death of Mr. Bourgeois for the Temple Trustees to carry the drawings further. This matter was put in charge of The Research Service of Washington, D. C., which allotted to our studio the development of the ornamental dome.

I can not begin to tell you how many factors enter into such a problem and I am sure that we automatically give consideration to many

without being able to recall or to name them: Just as an operating surgeon might know the position and function of every vein and sinew, the names of which have long since been forgotten. So in discussing such a problem consideration can be given only to principles such as these: The decoration of the temple must always be subservient to the architecture, the theme of the ornament must not be lost. The craftsmanship must be adequate, practical and economi; The materials must be suitable and enduring.

Were we to treat the exterior surface of the dome so that the perforations were too large they would destroy the architectural continuity. Were they too small they would not appear to be perforations. If the surface were simply perforated without further treatment the decoration would be inadequate, the theme would be lost, there would be no pathways of the stars nor movements of living things. All this must be modelled into the surface of the dome with care and good judgment, so, that at no place will the intertwining of this complicated grille escape from the configuration of the hemisphere. The interior surface of the dome is the subject of another group of considerations. If the solids between the perforations are too large the dome will appear as a dark surface spotted with bright dots. It would be like looking into a colander. If the solids be too thin, the light which enters will seem to bend around them and the bright spots will resolve into a confused blur. The pattern would be lost. And so with time and the greatest of care every ornamental detail must be adjusted to the unity of the architecture and the sequence of the story, as words are made to tell a story in the cadence of a poem.

Intermediate between the artistic and the practical there is a zone of translation where the asthetic is translated into the practical and where the complex is made simple. One who makes this translation must be thoroughly versed in theory and in practice. He must be able to understand the abstract form of a project and the means by which it may be determined in material by the operation of craftsmen. Such a translation has a real economic value, for it brings to the execution of the work many pairs of skillful hands which would not be available if the pure form of the project were not determined in the medium of the craftsmen. This work was undertaken by my associate, Mr. Taylor. It required a perfect understanding of all the factors of the problem and unusual resourcefulness. It was necessary to express the forms, relations and measurements in terms which our craftsmen could understand and use. In my opinion it was one of the most important factors in the execution of the work. Imagine translating such a theme into a practical operation, which would not involve anything new in the

FIG. 1, 2 AND 3

Fig. 1—"We began our model by constructing in the yard of our studio a full-sized wooden frame representing exactly the steel for one ninth of the dome." Fig. 2 and 3—"Every available shred of information about the structure we reproduced at full size on specially prepared strips of concrete pavement."

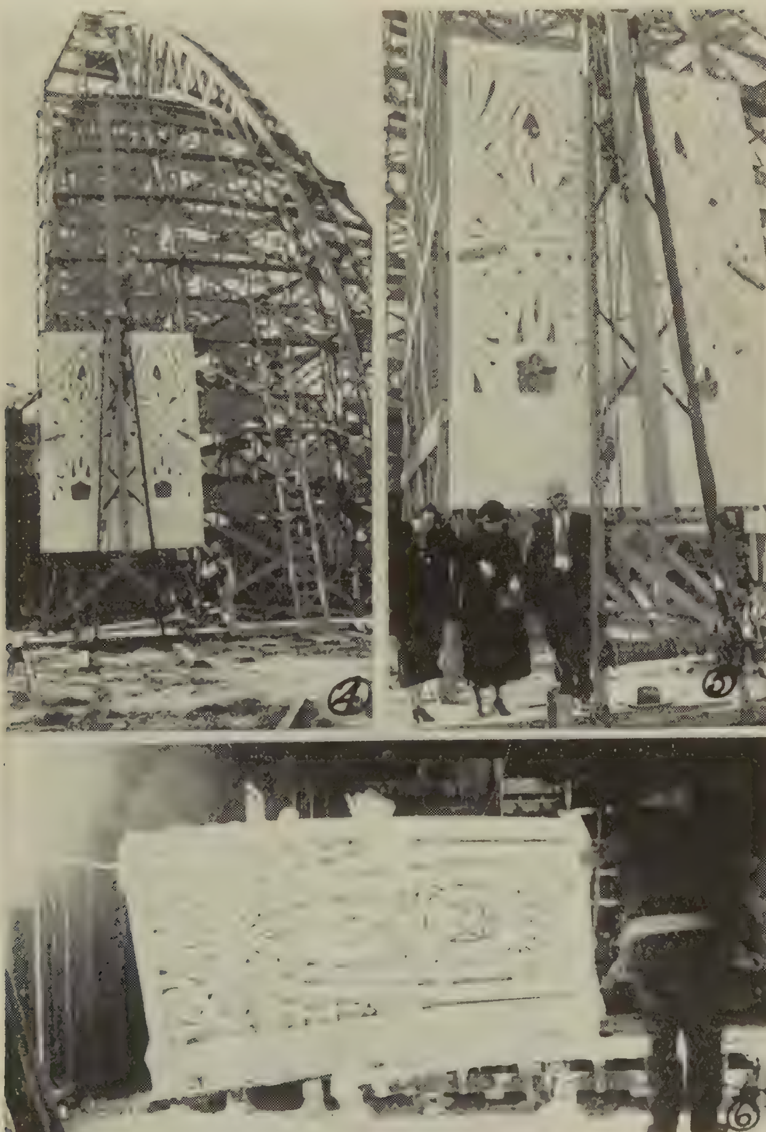

FIG. 4, 5 AND 6

Fig. 4—"A frame was made to represent the form of the great ribs." Fig. 5—"The precast sections of the field were joined in a pattern following closely the center lines of the steel ribs and purlins." Fig. 6—"The great ribs we divided into voussoirs corresponding to the heights of the sections of the field from purlin to purlin."

technique of the studio. Spherical measurements must become bits of wood of certain length cut to a given radius, complicated angles became jointed boards, skewed solids became simple frame work with internal bracing. The pathways of the stars were just clay models of ornamental grilles, plaster casts were just plaster casts and piece molds remained unchanged. The work of the craftsmen was as yesterday and the day defore. It interested them. It was in their medium. They understood it. Formulas became pieces of wood and of plaster: We have men who can do nothing with formulas but who can do wonders in wood and in plaster. This translation was brought about by means of a full-sized model of one ninth, forty degrees, of the dome. In my opinion a model of this kind was practically necessary. Drawings could have been made but from the viewpoint of the studio they would have been exceedingly complicated. Certain architectural requirements disturbed the geometrical symetry of the dome. By this I mean that the great ribs would not have been well done if they radiated simply from the pole as do meridians. They needed a thickness at the top which moved their sides from the meridians of the sphere and required that they be joined together at the top into a boss. Further, the sides of these great ribs warp continuously from the spring line of the dome where they are radial, to the top where they are parallel. Cross sections through the rib taken at intervals between the bottom and the top may be likened to a series of trapedzoids becoming more nearly rectilineal as they progress from the bottom to the top. In addition to this all fillets diminish at a purely arbitrary rate from bottom to top and the fields of the ornament are continuously changing. Theoretically the three panels in the field of the dome were distorted but the distortion was actually very little and practical economy indicated that they should be made symetrical to a degree where one model and one mold would serve for all.

We began our model by constructing in the yard of our studio a full-sized wooden frame representing exactly the steel for one ninth, of the dome. To do this we gathered from the structural drawings of the building all available information pertaining to the dome. All of this was condensed to one diagram showing a reflected plan and a section. This was a diagram pure and simple, there was nothing pictorial about it. It contained every available shred of information about the structure and we reproduced it at full-size on specially prepared strips of concrete pavement in the yard of the studio. The principal lines of the plan were extended far out beyond the periphery of the dome to points where one could see up and over the dome. These lines terminated in bronze pins set in concrete hubs. Over the plan

a frame work was constructed and on it were placed timbers located exactly as are the steel ribs and purlins on the skeleton of the dome. From over the pins, which terminated the principle lines of the plan, a transit quickly and easily drew planes up and through the curved surfaces of the dome, just as a great invisible knife might slice a melon. Strips of wood were made which represented the thickness of the concrete shell and a frame was made which represented the form of the great ribs. These were carefully lifted up over the frame work of the dome and carefully set with the aid of the transit in their proper relation to the ornamental dome and the structural steel.

Now for the first time, we faced reality and were able to see the relation between the existing steel structure and a proposed concrete covering. All other relations of form such as that between the area of the concrete dome and it's thickness; the relations of length, height and width in the great ribs, and curve of the rib up over the arc of the dome, all such ceased to be concepts and became experiences. The question whether this dome should be poured in place as a continuous fabric or precast and set, ceased to be a question. It was immediately apparent to practical judgment that a perforated concrete shell such as this dome, if cast as a continuous fabric attached to and supported by steel members, would tear itself to pieces in its first drying. I do not mean by this that the dome would break into many pieces and fall to the ground but I mean that the first volume change between wet and dry would set up internal stresses which could be relieved only by a great number of incipient cracks which might heal or which might grow larger as time passed. We, therefore, decided to precast the dome and set it in place. In doing this we completely excluced every element of construction, even a stiffening effect and placed the concrete dome simply as a load upon the steel. We decided that the precast sections of the field might each contain a hundred square feet more or less and that they might be jointed in a pattern following closely that established by the center lines of the steel ribs and purlins. The great concrete ribs we divided into voussoirs the length of which corresponded to the heights of the sections of the field from purlin to purlin. Pieces of this size would, we estimated, weigh approximately two or three tons. They would be large enough to contain a dignified section of ornament and small enough to be easily handled and to be reinforced against shrinkage with a reasonable hope of success. Between each precast section there is an open joint one half inch wide. It is provided to allow free movement in every direction to each cast, and to provide a means of taking up such variations as would naturally occur between the contours of the steel skeleton and the concrete shell.

Considered from the point of view of appearance such joints in the surface of a white dome should be hardly visible. On a curved surface one quickly looses direct elevation and in perspective the joints would quickly be lost. A brilliantly white surface may be expected to cast a halo of light over the joints to obscure them and further, if they do appear, they will be an orderly division of large areas that are well in scale with the dome. The pieces of the dome and ribs will be mechanically attached to the steel frame. Fittings will be cast in the concrete by means of which the castings will be bolted to the structural steel skeleton, nothing will be needed to set this dome in place but light hoisting apparatus and wrenches.

Much yet remains to be done in designing reinforcements, and in the selection of materials for the metal attachments, the reinforcements, the aggregate, the cement, and all the various details of preparation and execution. Each decision affords a new thrill and stirs our interest to the highest point. It is a project for which we feel that the best is none too good. It has never lost its interest for ourselves or for our men. Many delightful little stories of personal interest might be told of such incidents as these: My associate, Mr. Taylor personally laid out every line and measurement on the job; our plaster carver desires above all else to carve the great plaster model by himself without help; a member of the Baha'i Faith wanted to give all the the aggregate, if the quartz deposit on his homestead would meet our requirements, Unfortunately it did not. And so the development of the work goes on. It has been sincerely studied and sincerely met. No combination of steel and concrete could be more frankly made. No separation of finish from structure could be more completely made. I am deeply impressed by the simplicity and the economy of this solution to a complex problem, and I present it to you for your consideration.

I have spoken to the Institute before of a method of construction, which completely separated the structure and the ornamentation, pointing out its practical advantages and the reasons for them, indicating its history and giving examples of its application. The idea of building without decoration and of decorating after construction is not new. Indeed it is so very old that perhaps it may seem to be new. Familiarity with architectural form does not go back much further that the Renaissance and the same may be said of building methods, therefore, very old forms and very old methods particularly when applied by a new technique to a new material may easily be regarded as a daring innovation.

When the time came to build, the Temple Trustees were forced to decide whether the temple should be built as an indeterminate structure or whether it should be translated into a conventional form. I do not know the reasoning which led them to accept the conventional design. We have never considered the structure of the building except to dream that this Temple might have been built as it seems to be built. On the other hand I am sure that sound economic reasoning led them to decide to separate completely the ornamentation from the structure. The separation was as complete as it could possibly be. The structural elements were entirely completed before we began our work which consisted only in clothing the skeleton with an ornamental covering expressive not only of the form but the spirit of the architect's design.

A third paper, on Baha'i Temple presented at the 29th Annual Convention by Benjamin Shapiro, descriptive of the structural design of the building which Mr. Earley will decorate, is not available at this time. It is tentatively scheduled for publication in October 1933 (Proceedings, Vol. 30) and discussion of the work will then be invited.

Cement Investigations for the Hoover Dam*

BY RAYMOND E. DAVIS, R. W. CARLSON, G. E. TROXELL AND J. W. KELLY†

INTRODUCTION

This paper briefly describes investigations now in progress, principally in the Engineering Materials Laboratory of the University of California at Berkeley, as a basis for selecting a cement having most favorable chemical and physical properties for the construction of the Hoover Dam. The investigations are being conducted under a cooperative agreement between the University of California and the United States Bureau of Reclamation, with the additional cooperation of the California Portland Cement Co., Monolith Portland Cement Co., Portland Cement Association, Riverside Cement Co., Southwestern Portland Cement Co., and Yosemite Portland Cement Corp. The object of the tests is to determine the influence of chemical composition, fineness of grinding, and method of manufacture of cement upon heat of hydration, strength, volume changes, and durability of mortars and concretes. This report summarizes the major results so far obtained and presents tentative conclusions.

The hardening of portland-cement concrete is a chemical phenomenon, and the hydration of the cement is an exothermic reaction. When water is available for the hydration process, it appears that the rapidity of hydration depends upon the chemical composition of the cement, the size of the cement particles, and the temperature of the concrete of which the cement is a component part, and that both the rate of gain in strength and the rate at which heat is generated during the hardening process are functions of the rapidity of hydration.

As will be shown later, the strength, heat of hydration and other properties exhibited by a cement are materially influenced by its chemical composition. The works of Bogue and others indicate that portland cement is made up of four major compounds—tricalcium silicate (herein designated as C_3S), dicalcium silicate (C_2S), tricalcium aluminate (C_3A), and tetracalcium alumino-ferrite (C_4AF). Given

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the oxide composition, the quantities of the four compounds above named may be calculated by methods described by Bogue. It is generally believed that the magnesium oxide present in portland cement remains more or less inert, so far as its effect upon strength is concerned, and it is not considered to combine chemically with other oxides present.

In discussing the effect of chemical composition, it has been found convenient to express chemical influences in terms of these major calculated compounds. Among commercial cements, the percentages of the four compounds vary greatly, but on the average, the sum of the tricalcium silicate and dicalcium silicate (the strength-giving compounds) is perhaps 75 per cent of the total finished cement, while the sum of the tricalcium aluminate and tetracalcium alumino-ferrite (compounds which add little, if anything, to the strength) is perhaps 20 per cent.

It has been shown that the effect of tricalcium silicate is to produce early strength and that the effect of dicalcium silicate is to produce gradually increasing strength over a long period. By the proper manipulation of raw materials, the ratio of tricalcium silicate to dicalcium silicate may be made to extend over a wide range, and thus, other things being equal, there may be produced cements which harden quickly or cements which harden slowly. Among the ordinary commercial cements which have been or are now being manufactured, the tricalcium silicate content varies from perhaps 20 to 70 per cent, and the dicalcium silicate content varies from perhaps 5 to 50 per cent. Portland cements which are high in tricalcium silicate are spoken of as "high-lime" cements, and those which are high in dicalcium silicate as "high-silica" or "low-lime" cements.

Also, by manipulation of raw materials, the ratio of tricalcium aluminate to tetracalcium alumino-ferrite may be made to extend over a wide range. Among ordinary commercial cements, both the tricalcium aluminate and the tetracalcium alumino-ferrite contents vary roughly from 5 to 15 per cent.

The sizes of cement particles have an important influence upon the physical properties of concrete, but it has been found that the ordinary method of designating the fineness of cement by means of the percentage passing the 200-mesh sieve is totally inadequate. For cements, just as for concrete aggregates, a true picture can only be obtained through a size-distribution curve. The "specific surface", or calculated surface area of cement particles in square centimeters per gram, provides an index to the fineness, which, while not giving all the information shown by a particle-size distribution curve, yet

is a vastly more reliable index to fineness than is the percentage passing the 200-mesh sieve. By way of illustrating the inadequacy of the present standard method of determining fineness of cement, the relation between the specific surface of a cement and the percentage passing the 200-mesh sieve might be likened to the relation which exists between the fineness modulus of a graded aggregate of maximum size 3 in. and the percentage passing the 2-in. sieve. Inasmuch as the hydration of a cement particle must proceed from the surface, it is clear that the activity of a given quantity of cement should be more or less related to the surface area of its particles.

While among the commercial cements at present manufactured, the fineness as measured by the percentage passing the 200-mesh sieve varies from perhaps 84 to 98, the corresponding range of fineness as measured by the specific surface in square centimeters per gram is perhaps from 1100 to 1900 (Fig. 2). In general, in the discussions which follow, fineness of cement will be designated in terms of specific surface.

PROGRAM OF INVESTIGATION

The general program of cement investigation for the Hoover Dam includes tests on more than 15,000 specimens, using a total of 93 cements, as indicated below:

1. 48 cements of 16 commercial brands, some of which are commercially ground to various degrees of fineness and some laboratory ground to a fixed fineness—clinker of 4 brands heat-treated.

2. 45 laboratory cements covering an extreme range of chemical composition and fineness—clinker of 3 compositions heat-treated by each of two processes.

Tests on mortars and concretes employing the 93 cements mentioned above are in progress or have been completed for the purpose of determining the effect of chemical composition, fineness of grinding, and heat treatment of clinker upon the heat of hydration, strength, durability, and volume changes of mortars and concretes. The work has involved the production and preparation of cements for test, and carrying out a test program for each cement, as follows:

1. Chemical analysis of cement clinker and finished cement.

2. Fineness as determined by 200-mesh sieve, Pearson air analyzer, and Riverside microneter.

3. Standard A.S.T.M. physical tests for normal consistency, time of setting, soundness, and tensile strength.

4. Specific gravity.

5. Water-cement ratio required to produce a fixed consistency of concrete. (For each cement, the water-cement ratio thus determined was used for all specimens.)

6. Calorimetric test on large concrete cylinder of Hoover Dam mix, to determine the approximate rate and amount of heat generation. (The time-temperature relation thus determined was used in curing specimens for other tests.)

7. Heat-of-solution tests on unhydrated ("dry") cements and partially hydrated cement pastes for various periods of hydration up to one year, to determine accurately the rate and total amount of heat generation under mass-concrete curing conditions.

8. Adiabatic calorimetric tests (for selected cements only) on large concrete cylinders, to determine the rate and total amount of heat generation; used as a control and check for the heat-of-solution tests.

9. Compression tests on 2 by 4-in. cylinders made from standard mortar and cured under standard moist conditions at 70°F.

10. Compression tests on 2 by 4-in. mortar cylinders of 1:3¼ mix, using Hoover Dam sand, cured under each of the following conditions:

a. "*70°F.-curing*", in unsealed metal containers under moist conditions at 70°F.

b. "*Mass-curing*", in sealed metal containers under temperature and moisture conditions calculated to be the same as within mass concrete for the particular cement.

11. Tests to determine durability of mortar containing Hoover Dam sand by determination of compressive strength after repeated alternations of high and low temperature and alternations of wetting and drying.

12. Measurements to determine volume changes of 1½ by 1½ by 12-in. mortar bars of 1:3¼ mix using Hoover Dam sand, mass-cured for 28 days, after which age half of the bars are stored continuously under water at 70°F. and the remainder are stored continuously in air of 50 per cent relative humidity at 70°F.

13. Tests to determine the absorption of water by neat-cement paste, both under constant-temperature (70°F.) curing conditions and under mass-concrete curing conditions.

Further, auxiliary tests are being made on selceted cements to determine the effect of variations in curing temperature at various stages of curing upon the properties under investigation, the relation of mortar strength to concrete strength, the relation of particle size of cement to heat of hydration, the effect of water-cement ratio upon heat of hydration, etc.

TESTING APPARATUS AND TESTING METHODS

These studies have involved the use of unusual apparatus and methods of testing, particularly with regard to the curing of specimens

under variable conditions of temperature and humidity, and with regard to the determination of the heat of hydration. A condensed description of apparatus and methods follows:

1. *High-insulation calorimeters*, heavily insulated, housed in constant-temperature (90°F.) room; concrete specimens containing electrical resistance thermometers; inert concrete specimen to provide correction for heat transfer; used for approximate determination of heat generation up to age of 7 days.

2. *Variable-temperature chambers*, with automatic or manual control, for curing specimens under desired time-temperature conditions.

3. *Humidity control room*, with atmosphere of 50 per cent relative humidity at 70°F., for storage of volume-change bars.

4. *Adiabatic calorimeters*, with automatic or manual control; highly insulated chambers equipped with means for regulating temperature to within ± 0.1°F. or ± 0.5°F. of that of concrete specimen; thus, the rise of temperature of a hydrating specimen is a measure of the heat of hydration.

5. *Heat-of-solution calorimeter*, for determining the heat of solution of dry cement and of partially hydrated cement paste, the difference between these two values giving the heat of hydration up to the age of test. Calorimeter vessel equipped with electrical heater coil, mechanical stirrer, and electrical resistance thermometer, and placed in constant-temperature bath. Sample of cement dissolved in a mixture of nitric and hydrofluoric acids in calorimeter vessel, and temperature rise converted into heat units, expressed in calories per gram of cement. Use economical as compared with adiabatic calorimeters. Results equivalent to those obtained from temperature measurements on adiabatically cured concrete, provided curing temperatures are the same.

6. *Refrigeration room*, precisely maintained at 23°F., for storage of durability specimens during their freezing period in each cycle of treatment.

7. *Dry chamber*, with hot-air blast maintained constant at 160°F., for storage of durability specimens during their drying period in each cycle of treatment.

8. *Microneter*, for determining fineness of cement, operating on the sedimentation principle. Essential parts are (1) vertical cylindrical settling tube, in which the sample of cement is placed and mixed with the settling medium (distillate), (2) a sensitive pressure indicator connected to the settling tube near the base, (3) photographic means for recording the variations in pressure during settling. Employing Stokes' Law, the particle-size distribution is calculated from the time-pressure relation.

HEAT OF HYDRATION OF CEMENT

It has already been pointed out that the hardening of cement-water mixtures is a result of the chemical combination of cement and water. It may be stated further that as the cement hydrates each of the four major compounds of the cement combines with the water, more or less independently. Since it has been found that each of the separate compounds which are present in cement when combining with water liberates different amounts of heat, it might be expected that the heat of hydration of cement would vary with the percentages of the individual compounds present. The tests here discussed prove this to be the case and go a step farther in showing to what extent each of the principal compounds in cement influences the heat of hydration for the conditions under which the cement is used in service.

When considering the effect of any one factor, such as chemical composition, upon the heat of hydration of cement, it is helpful to bear in mind all other factors which

have appreciable influences. The principal factors affecting heat of hydration of cement during any period of hydration are as follows:

1. Chemical composition.
2. Fineness and particle-size distribution.
3. Water-cement ratio.
4. Temperature conditions during hydration.

To investigate the effect of chemical composition of cement upon the heat of hydration, tests have been made on a large number of commercial and laboratory cements for which chemical compositions differ widely, while all other factors are either maintained constant or allowed to vary in some prescribed manner representative of service conditions, all as previously described.

Under the conditions of the tests, without exceeding limits of compositions practicable of commercial manufacture and with ordinary values of loss on ignition, insoluble residue and free lime, large differences in heat of hydration have been observed. In Table 1, maximum, minimum and average values of observed heats of hydration for ages of 7, 28 and 90 days are presented to show in a general way the extent to which chemical composition influences heat of hydration. The units employed are calories per gram of cement, which can readily be converted into equivalent temperature rise of a particular concrete by applying a suitable factor. This factor is convenient to remember for concrete made of ordinary siliceous aggregates with 6 sacks of cement per cubic yard, since if no heat escapes one calorie per gram corresponds to a temperature rise of approximately one degree Fahrenheit. Thus for concrete of 4 sacks of cement per cu. yd., as proposed for the Hoover Dam, one calorie per gram corresponds to a temperature rise of approximately two thirds of one degree Fahrenheit, provided that no heat escapes.

TABLE 1—RANGE OF HEAT OF HYDRATION FOR LABORATORY CEMENTS OF SPECIFIC SURFACE 1200 SQ. CM. PER GRAM

Range	Heat of Hydration, calories per gram of cement		
	7 da.	28 da.	90 da.
Maximum	117	121	122
Minimum	25	39	57
Average	78	91	97

From the values in Table 1, it is apparent that large differences in heat of hydration can be produced by altering the chemical composition.

In Table 2 are more complete data for a number of selected cements which are representative of high-heat, normal, and low-heat compositions. The differences in heat of hydration are relatively much greater than are the corresponding differences in oxide composition (not shown) of the cements. This lends weight to the assumption that compounds are present in cement, the proportions of which are changed to a comparatively large extent by a small change in oxide composition.

When compound composition is considered with relation to heat generation, it is found that each compound contributes a definite share of the total heat of hydration. In a general way, the total heat released by a unit weight of any of the compounds is proportional to the rate at which the compound hydrates. Thus tricalcium aluminate, which hydrates to a large extent in a single day, releases the largest total amount of heat per unit weight of any of the compounds. Tricalcium silicate, which hydrates to a large extent within the first week, releases the next largest amount of heat per unit

weight. Dicalcium silicate and tetracalcium alumino-ferrite, both of which hydrate slowly, release correspondingly small amounts of heat.

TABLE 2—EFFECT OF CHEMICAL COMPOSITION UPON HEAT OF HYDRATION AND MORTAR COMPRESSIVE STRENGTH OF SELECTED LABORATORY CEMENTS OF SPECIFIC SURFACE 1200 SQ. CM. PER GRAM

Cement No.	Compound Composition, per cent				Heat of Hydration (Mass Curing), calories per gram of cement			Compressive Strength, p. s. i.					
								Mass Curing			Curing at 70°F.		
	C_3S	C_2S	C_3A	C_4AF	7 da.	28 da.	3 mo.	7 da.	28 da.	3 mo.[a]	7 da.	28 da.	3 mo.
L-2	68	9	12	9	100	110	111	4320	5110	5130	3790	5730	6260
L-1	55	23	11	8	86	99	104	4080	5070	5480	2470	4300	5740
L-3	26	52	11	8	71	86	96	2150	4550	4770	1180	3380	5120
L-4	12	67	10	9	50	68	77	900	2620	3320	600	1660	3260
L-5	51	25	18	3	115	121	122	3810	4020	3890	2590	3940	4570
L-6	55	24	0	18	75	86	89	4290	5740	5820	2330	3700	5600
L-7	12	70	0	15	28	45	61	610	2320	4110	280	1370	3800
L-20	31	49	3	15	56	72	81	1900	4010	5130	1360	2610	5020

[a]Mass curing for 28 days, then curing at 70°F. until test.

The experimental data have been subjected to a mathematical analysis to find the most probable contribution to heat of hydration of each of the four principal compounds in cement at each of the ages, 7, 28, and 90 days. The analysis is based on the assumption that each compound contributes heat in proportion to the percentage of that compound present. It is known that this assumption can not be strictly correct, especially since, for reasons previously stated, water-cement ratios and curing temperatures are neither maintained constant nor are they in proportion to the amounts of each compound. However, the effect of these variations is believed to be reasonably small, and the consistency with which the derived factors check actual observed values has proven that the analysis is satisfactory for practical purposes.

In Table 3 is given the amount of heat contributed by one per cent of each compound up to each of the ages, 7, 28, and 90 days. It is observed that the heat released by the hydration of one per cent of C_3S is about one half of that released by an equal amount of C_3A. The small but definite increase in heat-of-hydration factors from 7 to 28 days is believed to signify that the hydration of C_3S, although nearly complete at 7 days, is continuing at a very slow rate after the age of 7 days.

The heat contributed by each per cent of C_2S is shown to be only 0.2 calories per gram up to the age of 7 days, but at 90 days it is 0.6 calories per gram. This signifies that only a small percentage of this compound hydrates prior to the age of 7 days. There is little doubt but that the liberation of heat continues at a gradually reducing rate even after the age of 90 days. Results obtained by extrapolating heat-of-hydration curves, however, indicate that the amount of heat released after 90 days is large enough to be important only for cements extremely high in C_2S.

Of all the major compounds in cement, C_4AF appears to contribute least to heat of hydration. Referring to Table 3, it may be seen that the factors for this compound are almost negligible at all ages. It should be borne in mind that the factors in Table 3 are for mass-curing conditions, these being unusual in that the curing temperature depends upon the amount of heat liberated, and in that the water-cement ratio depends upon the water required to produce a fixed consistency of concrete. This probably explains why the factors for C_4AF are smaller than are those obtained for this compound by other investigators. For any conditions, however, the heat contributed by C_4AF is relatively small.

TABLE 3—CONTRIBUTION OF MAJOR COMPOUNDS IN CEMENT TO HEAT OF HYDRATION FOR LABORATORY CEMENTS OF SPECIFIC SURFACE 1200 SQ. CM. PER GRAM

Age, Days	Contribution of Each Per Cent of Compound to Heat of Hydration, calories per gram			
	C_3S	C_2S	C_3A	C_4AF
7	1.1	0.2	2.4	0.0
28	1.2	0.4	2.3	0.1
90	1.2	0.6	2.2	0.1

The small contribution of C_4AF and the large contribution of C_3A to the heat of hydration suggests the desirability, when preparing a cement for mass-concrete construction, of reducing the heat of hydration by decreasing the percentage of C_3A with a corresponding increase in C_4AF. A further reduction in heat of hydration may be obtained by decreasing the percentage of C_3S with a corresponding increase in C_2S. In the latter case, as will be shown later, the reduction in heat of hydration is at the expense of early strength.

The data in Table 3, although obtained from an analysis of laboratory cements, have been found to apply almost as well to commercial cements of equal specific surface (1200 sq. cm. per gram). The only notable exceptions are for commercial cements which have been aged so that the loss on ignition is high. In these cases the observed heats of hydration are lower than those calculated from the factors in Table 3. Fair agreement is obtained by assuming that the heat of hydration is decreased 6 calories per gram for each per cent loss on ignition. It appears reasonable that the prehydration caused by the water or carbon dioxide which makes up the loss on ignition should be of this order of magnitude.

Effect of fineness of cement upon heat of hydration

The effect of fineness of cement upon heat of hydration has been investigated for 3 commercial cements and 3 laboratory cements of a wide range of chemical composition, each ground to several degrees of fineness. It is found that the effect of fineness is neither the same for all ages nor the same for all chemical compositions. However, within the limits of the tests, for a given composition, increase in heat of hydration is directly proportional to increase in fineness, expressed in terms of specific surface. This relation permits the determination of an average coefficient for each cement composition at each age, this coefficient being defined as the change in heat of hydration in calories per gram due to a change in specific surface of 100 sq. cm. per gram. The coefficients for the six cement compositions investigated are found to decrease with age, the average value for 1 day being 3.2 calories per 100 sq. cm., for 28 days 1.8 calories per 100 sq. cm., and for 90 days only 1.5 calories per 100 sq. cm. That is to say, the effect of fineness upon heat of hydration generally diminishes with age of hydration. It is significant that for the ages of 28 and 90 days the effect is comparatively small, since a change in specific surface of about 10 per cent causes a change in heat of hydration of only about 2 per cent.

As previously stated, the effect of fineness upon heat of hydration of cement varies with the chemical composition. The presence of high percentages of C_3A tends to increase the effect of fineness for early ages and to decrease it for later ages, while the presence of high percentages of C_2S acts in the opposite direction.

The approximate effect of fineness upon the heat of hydration of the average present-day commercial cement is for the ages of 1 and 90 days, 4.5 and 1.2 calories per 100 sq. cm., respectively. It is evident that while the effect for the age of 1 day

is appreciable, the effect for the age of 90 days is too small to have much practical significance.

Effect of water-cement ratio upon heat of hydration

It is already well known that an increase in the water-cement ratio of plastic concrete decreases its strength, watertightness and weather resistance, and increases the amount of shrinkage upon drying. It is now found that an increase in the water-cement ratio increases the heat of hydration of cement. Within the working range of concrete consistencies, an increase of 0.01 in the water-cement ratio by weight adds on the average about 0.3 calories per gram to the heat of hydration up to the age of 7 days, and about 0.4 calories per gram up to the age of 28 days, with little change thereafter.

There appears to be no definite relation between the chemical composition of the cement used and the effect of water-cement ratio upon the heat of hydration.

Effect of richness of mix upon heat of hydration

Numerous tests on neat-cement specimens cured according to the time-temperature conditions for corresponding mass concrete have shown heats of hydration in excellent agreement with those obtained directly from the concrete itself by means of adiabatic calorimeters. It may be stated that cement when mixed neat with a given amount of water liberates practically the same amount of heat as does the same cement when mixed with various amounts of aggregate and the same amount of water, provided that the curing temperatures are identical.

From a consideration of the effect of high curing temperatures, it may also be stated that neat-cement samples cured with small heat losses (and therefore extreme rises in temperature) will ultimately generate less heat than will the same samples cured under temperature conditions such as obtain within mass concrete, although the reverse may be true within the first day or so.

RELATION BETWEEN HEAT OF HYDRATION OF CEMENT AND COMPRESSIVE STRENGTH OF MORTAR

In mass concrete, since it is generally desired to secure concrete of ample strength with the minimum development of heat, strength and heat are important as related one to the other. For this reason the ratio of mortar strength to heat of hydration (here designated as the strength-heat ratio) is useful, since it designates the relation between these two properties in a single number. The strength-heat ratio is expressed in terms of pounds per square inch per calorie per gram. Other things being equal, the higher the strength-heat ratio the more desirable the cement for mass concrete.

Effect of chemical composition of cement upon strength-heat ratio

Examples demonstrating the principal relations between chemical composition, heat of hydration, and strength are given in Table 2. Studies of the more complete data show that the effect of increasing the percentage of C_3A in a cement is to decrease considerably the strength-heat ratio at all ages, while the effect of increasing the percentage of C_4AF is to decrease the ratio only slightly. This is illustrated through a comparison of the values given in the table for cements L-5 and L-6, which differ mainly in percentage of C_3A and C_4AF.

While both C_3A and C_4AF tend to decrease the strength-heat ratio, C_3S and C_2S tend to increase it. Studies indicate that for ages of 28 days and greater each per cent of C_3S contributes 0.65 to the strength-heat ratio, while each per cent of C_2S contributes 0.75 to the strength-heat ratio.

TABLE 4—EFFECT OF FINENESS OF CEMENT UPON HEAT OF HYDRATION AND MORTAR COMPRESSIVE STRENGTH

Cement No.	Compound Composition, %				% Pass 200-Mesh	% 0-20 Microns	Spec. Surface, cm²/g.	Water[a] for Normal Consistency, % by wt.	Concrete Water-Cement Ratio,[b] by wt.	Heat of Hydration, calories per gram Mass Curing			Compressive Strength, p. s. i.					
													Mass Curing			Curing at 70°F.		
	C_3S	C_2S	C_3A	C_4AF						7 da.	28 da.	3 mo.	7 da.	28 da.	3[c] mo.	7 da.	28 da.	3 mo.
30M	54	20	13	9	75.9	34.2	960	....	0.61	97	106	104	3160	3950	3740	1960	3710	4250
	57	16	13	9	85.3	39.2	1150	22.0	0.57	99	110	106	3830	3840	4390	2830	3740	4890
	61	14	13	9	92.5	50.6	1450	....	0.55	106	110	107	4610	4950	5440	4310	5380	6500
39E	43	30	6	11	80.8	38.4	1100	....	0.61	63	73	80	1600	2840	2970	1120	2240	3020
	42	31	8	10	89.6	42.4	1240	22.5	0.59	66	78	85	1920	3510	3290	1450	2970	3550
	44	29	7	11	97.0	55.1	1570	....	0.57	76	90	95	3750	5070	5450	2470	4390	5530
15M	39	36	12	9	75.6	41.0	1090	....	0.63	94	101	99	2380	3050	3240	1610	3100	3720
	38	37	13	7	85.9	43.2	1280	23.5	0.60	95	102	100	2830	3390	3380	2160	3000	4040
	39	34	11	9	94.2	57.8	1640	....	0.59	95	104	101	4100	4800	5130	3450	5000	5630
L-4-2	11	67	11	8	78.5	35.2	970	22.0	0.63	45	58	76	490	1540	2210	290	880	2220
					88.5	42.6	1130	23.6	0.60	48	65	80	550	1970	2990	340	1210	2860
					92.3	47.0	1330	23.8	0.59	51	69	83	680	2340	2830	410	1260	2870
					95.5	54.8	1540	25.0	0.59	55	74	85	620	2240	3360	410	1460	3580
L-13-2	57	25	8	7	81.1	37.2	980	21.5	0.60	81	95	102	2490	....	....	2190	3900	4910
					89.1	43.0	1210	22.2	0.59	91	102	107	3350	5010	4400	2840	4430	5230
					93.3	48.8	1370	22.8	0.56	99	110	113	4220	5720	5460	3310	4600	5240
					95.3	53.4	1550	22.8	0.55	100	111	114	4980	6280	6490	3600	4900	6240
L-20	31	49	3	15	77.9	36.0	920	20.0	0.58	50	66	77	2050	3930	4820	960	1980	3980
					87.1	42.6	1170	20.5	0.56	56	73	82	1900	4010	5130	1360	2610	5020
					92.7	48.6	1400	21.5	0.53	57	74	83	2100	4770	5730	1580	3220	5650
					95.3	54.8	1530	21.0	0.53	66	79	84	2770	5480	6590	1790	3560	5880
Average of 11 comm.	50	25	10	8	86.7	43.8	1200	22.8	0.59	86	98	100	2860	3480	3760	2170	3180	4140

[a]—A. S. T. M. standard.
[b]—Concrete water-cement ratio is that required to produce concrete of 140 per cent flow (24-in. flow table); cement-aggregate ratio 1:5.6 by weight, using 0 to ¾-in. Hoover Dam aggregate.
[c]—Mass curing for 28 da., then curing at 70°F. until test.

Effect of fineness upon strength-heat ratio

The effect of a given change in fineness upon strength has been found to be relatively several times greater than the effect upon heat of hydration. Calculations based upon the data of Table 4 indicate that the average effect of an increase in fineness of 100 sq. cm. per gram is to increase the strength-heat ratio by 5 per cent. Therefore, as far as the relation of strength to heat is concerned, substantial benefit is derived by increasing the fineness of a cement.

Effect of water-cement ratio upon strength-heat ratio

Since an increase in water-cement ratio both decreases strength and increases heat of hydration, the effect of variations in water content upon the strength-heat ratio is relatively large.

COMPRESSIVE STRENGTH OF MORTAR AND CONCRETE

Effect of chemical composition upon strength

The following discussion of the effect of chemical composition of cement upon the strength of mortar and concrete is based chiefly upon the results of tests on 20 laboratory cements ground to equal fineness (specific surface 1200 sq. cm. per gram), designed especially to determine the influence of chemical composition upon strength and other properties (see values for selected cements in Table 2). While on account of lack of space, the results of tests on commercial cements are not here presented, they are in satisfactory agreement with the values given here for laboratory cements. The results here under discussion are based upon compressive tests of mortars and concretes containing Hoover Dam aggregates.

Effect of tricalcium silicate and dicalcium silicate upon strength—Consulting Table 2, for the mass-curing condition, it will be seen that the cements which are high in tricalcium silicate (C_3S) content show high mortar strength at early ages, but relatively little gain in strength thereafter. For example, cement L-2 containing 68 per cent of C_3S has a mortar strength of 4320 p. s. i. at the age of 7 days, but only about 20 per cent greater strength at the age of 3 months.

Cements high in dicalcium silicate (C_2S) show under mass-curing conditions comparatively little mortar strength at early ages, but develop a substantial strength at later ages. For example, cement L-4 containing 67 per cent of C_2S has a mortar strength of only 900 p. s. i. at the age of 7 days but more than 3½ times this amount at 3 months. Between the ages of 28 days and 3 months, this cement exhibits an increase in mortar strength of more than 25 per cent.

As previously stated, the combined amount of C_3S and C_2S is roughly constant. Consequently, in general, a decrease in the amount of one means an increase in the other. In Fig. 1 is shown the effect upon strength, for conditions of curing at 70°F., of varying the ratio of C_3S to C_2S with other factors remaining constant. It will be noted here, as in Table 2, that cements high in C_3S show high early strength with small gains at later ages, while cements high in C_2S show low early strength but substantial increases in strength at later ages. It appears that the strengths, which differ considerably up to the ages shown, may be closer together or even equal at still later ages, since at the later ages the strength of cements high in C_2S content is increasing at a substantial rate.

Effect of tricalcium aluminate and tetracalcium alumino-ferrite upon strength.—From a study of data not presented here, it appears that tricalcium aluminate (C_3A) produces strength at the very early ages, that is, within the first day or so. However, the data of Table 2 indicate that in general at the later ages, the larger the C_3A con-

tent, the lower the strength. For example, comparing the strengths for cements L-5 and L-6 under conditions of mass curing, it is seen that even at the age of 7 days, the cement of higher C_3A content is of lower strength.

The apparent effect of C_3A upon strength is different for the two curing conditions, as will be shown later by the factors of Table 5. At 70°F., the temperature at which acceptance tests are usually made, at least up to the age of 28 days, greater strength is shown for the higher value of C_3A, while at later ages the reverse is true. Under mass-curing conditions, however, greater strength is shown for the higher value of C_3A only at the very early ages, and less strength is shown at ages of 28 days and greater. It thus appears that 28-day strength tests, under 70°F. curing, for cements of large C_3A content, cannot properly represent the strengths developed in mass concrete.

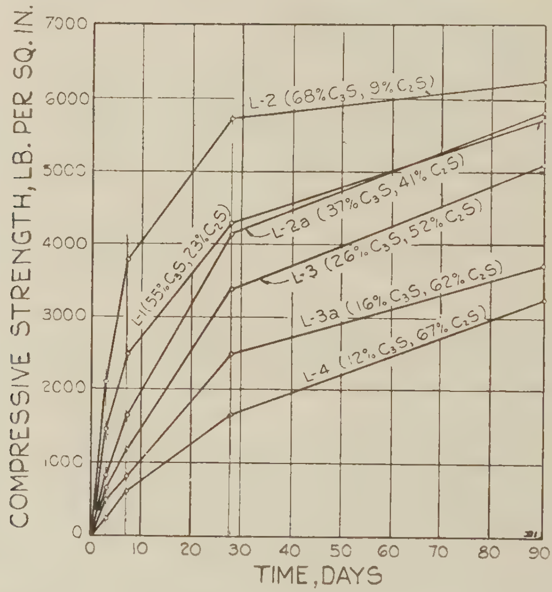

FIG. 1—EFFECT UPON STRENGTH OF VARYING THE RATIO OF C_3S TO C_2S, WITH C_3A AND C_4AF REMAINING CONSTANT. MORTAR SPECIMENS CURED AT 70° F.

Tetracalcium alumino-ferrite (C_4AF) appears to produce a moderate reduction in strength. While the reduction in strength is shown at all ages covered by the tests, its amount seems to be rapidly diminishing at the later ages, especially under mass-curing conditions, and there are indications that at still later ages this compound may even contribute to strength.

Effect of magnesia upon strength—The effect of magnesia (MgO) upon strength has been studied by comparison of test results for cements which have magnesia contents of 2.6, 4.7, 4.0 and 4.0 per cent, respectively, but which are otherwise similar. The variations in strength for either condition of curing are neither large nor consistent, indicating that, up to 5 per cent, the amount of magnesia has no considerable influence upon strength.

Mathematical study of contribution to strength of major compounds—The most probable contribution to mortar compressive strength of each per cent of the four major compounds in cement has been computed by the method of least squares for each of the ages, 7, 28 and 90 days, and for both mass and normal-temperature conditions of curing. The values thus obtained, based upon clinker analysis of laboratory cements of equal fineness, are given in Table 5, expressed in p. s. i. The contribution of the total percentage of each compound to compressive strength of mortar containing Hoover Dam sand (water-cement ratio approximately 0.57 by weight) may be determined for a given age and condition of curing by multiplying the percentage of that compound by the appropriate contribution value. The approximate total strength of the mortar may then be obtained by simply adding the contributions of the separate compounds. For example, the computed 7-day strength of 70°F.-cured mortar for cement L-1, which contains 55 per cent of C_3S, 23 per cent of C_2S, 11 per cent of C_3A, and 8 per cent of C_4AF, is $(55 \times 46) - (23 \times 1) + (11 \times 14) - (8 \times 11) = 2570$ p. s. i., which is in fairly close agreement with the experimental value of 2470 p. s. i.

TABLE 5—CONTRIBUTION OF MAJOR COMPOUNDS IN CEMENT TO MORTAR COMPRESSIVE STRENGTH FOR LABORATORY CEMENTS OF SPECIFIC SURFACE 1200 SQ. CM. PER GRAM

Curing Condition	Age, Days	Contribution of Each Per Cent of Compound to Compressive Strength, p. s. i.			
		C_3S	C_2S	C_3A	C_4AF
70°F.	7	+ 46	− 1	+ 14	− 11
70°F.	28	+ 76	+ 21	+ 12	− 57
70°F.	90	+ 87	+ 46	− 13	− 30
Mass	7	+ 73	+ 9	+ 3	− 52
Mass	28	+ 81	+ 33	− 14	− 28
Mass*	90	+ 77	+ 47	− 25	− 5

*Stored at 70°F. after 28 days mass curing.

For 20 laboratory cements of equal fineness, the average percentages of deviation of the computed values of strength from the observed values range from 6 to 14. Eliminating the values for several cements of extreme composition, the amounts of these average deviations are materially reduced.

Effect of fineness of cement upon strength

The data presented in Table 4 for both commercial and laboratory cements of a wide range of chemical composition show the characteristic effect of fineness upon the strength. In general, the finer the grinding the higher the strength, the percentage increase in strength at the age of 3 months being approximately ¾ of the percentage increase in specific surface of the cement. This advantage of finer grinding is noticeable for all cements and for all conditions of curing, and is largely maintained at the later ages, although the increase in strength due to finer grinding is at 90 days only about half that at 3 days. The fact that the water-cement ratios required to produce the desired consistency are lower for finer cement unquestionably accounts for a part of the difference between the mortar strengths for the fine and corresponding coarse cements. The curing conditions appear to have no appreciable effect upon the percentage of increase in strength resulting from an increase in fineness of the cement.

Comparison of mortar and concrete strengths

Investigations are in progress to determine the relation between the compressive strength of mortar and that of corresponding concrete of equal water-cement ratio, under mass curing conditions. The mortar specimens are 2 by 4-in. cylinders containing 1 part (by weight) of cement to 3¼ parts of 0 to No. 4 Hoover Dam sand; the concrete specimens are either (a) 6 by 12-in. cylinders containing 1 part of cement to 7 parts of 0 to 1½-in. Hoover Dam aggregate, or (b) 3 by 6-in. cylinders containing 1 part of cement to 5.2 parts of 0 to ¾-in. Hoover Dam aggregate. It is indicated that mortar having a water-cement ratio equal to that of concrete is on the average about one-third higher in strength.

EFFECT OF VARIATIONS IN CURING TEMPERATURE UPON HEAT OF HYDRATION AND STRENGTH

An outstanding feature of this investigation is the determination of the properties of mortar and concrete cured under variable temperature conditions. While the space available does not permit an extended discussion of the effect of variations in temperature upon the heat of hydration and compressive strength, a few relations may be stated here. Tables 2 and 4 contain some of the values upon which the statements are based.

For a cement of normal commercial composition and fineness, the strength of mass-cured mortar is, up to the age of 28 days, higher than that of mortar cured at 70°F., but at the age of 3 months the strengths are approximately equal. For cements of high dicalcium silicate content, the strength of mass-cured mortar is higher at all ages of test.

A condition of curing in sealed containers for 1 day at 70°F. and thereafter at 100°F. more nearly approaches the condition of mass curing than does curing continuously at 70°F. The 70-100°F. curing condition may be provided without the variable-temperature control equipment which is required for mass curing. For normal cements, the strengths obtained under this condition of curing are (after the age of about 3 days) somewhat lower than the strengths obtained under continuous mass curing, and the heats of hydration are slightly higher than for mass curing. For low-heat cements, the heats of hydration at the ages of 7 and 28 days are almost equal for the two curing conditions.

From the results of tests made to determine the effect of a wide range of curing conditions upon heat of hydration and strength, using 2 commercial cements (one having favorable qualities for mass-concrete construction), it appears that:

1. Up to the age of 3 months, higher strength, greater heat generation, and a greater strength-heat ratio are obtained by curing continuously at normal temperature (70°F.) than by curing either at low (40°F.) or high (110°F.) temperature. However, it appears that the strength for the low-temperature curing condition may ultimately equal or exceed the strength for normal-temperature curing.

2. Up to the age of 3 months, (a) continuous mass curing from a lower initial temperature results in slightly greater heat generation, and (b) continuous mass curing from a normal initial temperature (70°F.) results in higher strength than does continuous mass curing from either a low (44°F.) or high (102°F.) initial temperature. However, it appears that the strength for the condition of low initial temperature of mass curing will ultimately equal or exceed that for mass curing from normal initial temperature.

WATER-CEMENT RATIO OF CONCRETE OF FIXED CONSISTENCY

Effect of chemical composition of cement upon required water-cement ratio

It appears that the larger the C_3A content of a cement, the greater the amount of water required to produce constant consistency of concrete (an increase in the C_3A content of 5 per cent results in an increase of 0.01 in the water-cement ratio by weight). There is no evidence that variations in either the C_3S or C_2S content of a cement influence the water-requirement.

Since the range of C_3A content in commercial cements is small (approximately 5 to 15 per cent), the effect of variations in the chemical composition of cement upon the water-cement ratio of concrete is slight.

Effect of cement fineness upon required water-cement ratio

Within the range of fineness of normal cements, the finer a cement, the lower the water requirement to produce constant consistency of concrete (see Table 4). From the data available, it appears that an increase in specific surface of 100 sq. cm. per gram results in a decrease in the water-cement ratio of about 0.01 by weight (sufficient to account for an increase in strength of 3 per cent).

Relation between water requirement for normal consistency of neat-cement paste and water-cement ratio of concrete

For 76 of the cements included in this investigation, the average water-cement ratio required for 140 per cent flow of concrete (mix 1:5.6 by weight, using 0 to ¾-in. Hoover Dam aggregate) was 2.6 times that required for normal consistency of neat-cement paste, but among individual cements the value of this factor varied from 2.4 to 2.9. Further, it will be noted in Table 4 that, almost without exception, a *decrease* in the water-cement ratio of concrete is accompanied by an *increase* in the amount of water required for normal consistency. It therefore appears that normal-consistency tests are not a satisfactory indication of the relative water requirements of concretes.

MEASUREMENT OF CEMENT FINENESS

The fineness of each cement listed in Table 4 is indicated therein by each of three values, namely:

1. Per cent passing 200-mesh, determined by sieve analysis.
2. Per cent 0 to 20 microns in diameter, determined by means of the Pearson air analyzer.
3. Specific surface, in sq. cm. per gram, determined by means of the microneter.

Fig. 2 shows values of cement fineness as determined by 200-mesh sieve, air analyzer, and microneter. As might be expected, the correlation is poor between individual values of fineness as measured by sieve and as expressed by the specific surface. However, on the average, a fineness represented by 80 per cent passing the 200-mesh sieve corresponds roughly to a specific surface of 1000, 90 per cent to 1300, and 95 per cent to 1500 sq. cm. per gram.

A more consistent indication of the fineness (specific surface) of cement than that given by the 200-mesh sieve may be obtained by determining the percentage of particles from 0 to 20 microns in diameter, through the use of the air analyzer. The diagram of Fig. 2 shows that under the conditions of the tests, the specific surface was 28.6 times the percentage, by weight, of 0-20 micron particles. This constant would of course vary with apparatus and test conditions, but the point to be emphasized is that the percentage of 0-20 micron material as determined by the air analyzer seems to be an excellent index to the specific surface as determined through the use of the microneter.

The average fineness of 11 selected commercial cements, as shown in Table 4, is that represented by 87 per cent passing the standard No. 200 sieve, 44 per cent 0 to 20 microns, and a specific surface of 1200 sq. cm. per gram.

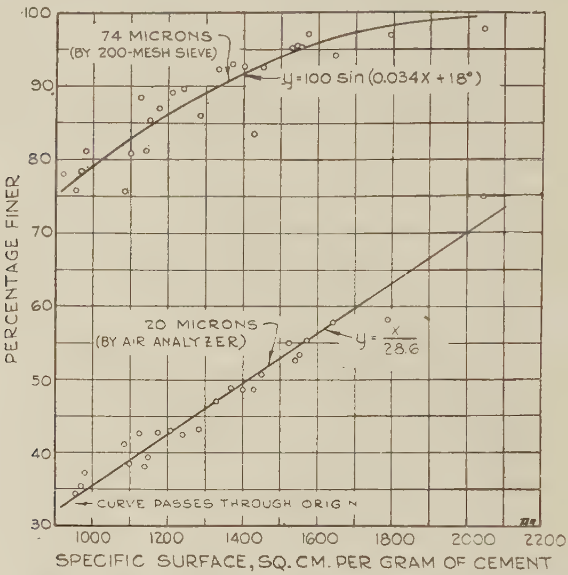

FIG. 2—RELATION BETWEEN CEMENT FINENESS, AS DETERMINED BY (A) 200-MESH SIEVE AND (B) AIR ANALYZER, AND SPECIFIC SURFACE, AS DETERMINED BY MICRONETER

TENTATIVE CONCLUSIONS

In the articles which follow will be submitted tentative conclusions based upon the results of tests to date.

In considering these tentative conclusions, it should be borne in mind that the tests are by no means finished, and that evidence or information which will later become available may serve to alter the opinions here expressed. Furthermore, the conclusions refer to cements which are manufactured under conditions in all respects similar to those under which normal portland cements are produced, and no attempt is made to pass upon the merits or demerits of compositions which fall outside the range of true portland cements.

1. The chemical composition of cement has an important effect upon both the rate and total amount of the heat of hydration during any period.

2. Tricalcium aluminate (C_3A) liberates more heat per unit of weight than does any other major compound, and this liberation very largely occurs during the early stage of the hardening process.

3. Tricalcium silicate (C_3S) is next in importance from the standpoint of heat liberated during the hydration process. Its heat is liberated principally during what may be called the "secondary" stage of hardening, which under normal curing conditions will be within the first week or even within the first day after hardening begins.

4. The compounds producing the least heat of hydration are dicalcium silicate (C_2S) and tetracalcium alumino-ferrite (C_4AF). The evidence is that the heat of hydration generated by these compounds is slow and long-continued.

5. The effect of an increase in the loss on ignition is a decrease in the heat of hydration. Approximately, this amounts to a decrease of 6 calories per gram of cement for each increase of 1 per cent in loss on ignition.

6. Other things being equal, the finer a cement, the greater the heat of hydration. For cements of high C_3A content, the difference between the heat of hydration of a coarse cement and the same cement finely ground is greater at the early ages than at the later ones; for cements of high C_2S content this difference is somewhat less at the early ages than at the later ones. For an average portland cement, an increase in specific surface of 100 sq. cm. per gram, under conditions of mass curing, will increase the 28-day heat of hydration approximately 2 calories per gram.

7. Increasing the water-cement ratio increases the heat of hydration, but the percentage of increase in heat of hydration is approximately one-fifth of the percentage of increase in water-cement ratio.

8. For the same water-cement ratio *and the same curing temperatures*, the heat of hydration of a given quantity of a cement is practically independent of the richness of mix, being essentially the same in a neat-cement paste as in a corresponding concrete.

9. Considering the ratio of strength expressed in p. s. i. to the heat of hydration expressed in calories per gram (which ratio is in one sense a measure of the efficiency of a cement), other things being equal, regardless of age, the higher the fineness the higher the strength-heat ratio; the lower the tricalcium aluminate content the higher the strength-heat ratio; and for the later ages, the higher the dicalcium silicate content the higher the strength-heat ratio.

10. The strength of concrete under both normal and mass curing conditions is affected greatly by the chemical composition of the cement.

11. Tricalcium silicate (C_3S) is principally responsible for the early strength of cement mortars and concretes. The indications are that

most of its contribution to the strength occurs within the first week of the hardening period.

12. Dicalcium silicate (C_2S) contributes very little to the early strength of cement mortars and concretes, but is responsible for most of the gain in strength after the first week. It provides a long-continued increase in strength so long as water is available for hydration.

13. Under conditions of mass curing, for those cements which are high in tricalcium silicate, there is little gain in strength after 28 days; but for cements which are high in dicalcium silicate, there is a large gain in strength after 28 days.

14. The dicalcium and tricalcium silicates, weight for weight, apparently contribute about equally to the ultimate strength.

15. Other things remaining equal, the greater the C_3A content and the less the C_4AF content, the greater the strength at the early ages but the less the strength at the later ages.

16. The effect of an increase in fineness is to increase the strength very materially, not only at the early ages but also at the later ages. The strength is increased nearly in proportion to the increase in the specific surface. This is in contrast to the effect of increase in fineness upon heat of hydration, which is relatively small.

17. Continuous mass curing results in higher strength at early ages than does curing at normal temperature. At later ages, mass curing results in higher strength than normal-temperature curing only for cements of high dicalcium silicate (C_2S) content.

18. For cements which are high in dicalcium silicate, the 7- and 28-day compressive strengths of mortars cured under normal conditions at 70°F. do not give a true indication of the strength of these mortars when cured under mass conditions. On the other hand, it has been determined that a curing condition of 1 day at 70°F. followed by a temperature of 100°F. until time of test produces results not greatly different from those which will be obtained under mass-curing conditions.

19. Other things being equal, the higher the C_3A content the greater the water requirement to produce a given consistency of concrete, although this increase in water requirement is not large. It is not apparent that variations in the amounts of the other major compounds appreciably affect the water requirement for concrete of given consistency.

20. Within the usual range of fineness of commercial cements, the finer the cement the less the water requirement to produce a concrete of fixed consistency.

21. There is no consistent relationship between the water required for normal consistency of neat-cement pastes and the water required to produce a given consistency of corresponding concretes.

22. The specific surface, as determined through the use of the microneter apparatus, is considered to be the most satisfactory measure of cement fineness. The percentage passing the 200-mesh sieve is not a satisfactory measure of fineness, but the percentage finer than 20 microns (determined by the air analyzer) is proportional, within fairly consistent limits, to the specific surface as determined by the microneter.

The foregoing paper is a brief summary only of research by the Bureau of Reclamation, which the Bureau proposes to publish in full. Announcement of that publication will be made in this JOURNAL. *Discussion is temporarily deferred until the publication of other reviews of Hoover Dam research as presented at the 29th Annual Institute Convention.*

Fifth Report on Column Tests at Lehigh University*

BY INGE LYSE†

SERIES 4—TESTS ON THE AMOUNT OF LOAD A REINFORCED CONCRETE COLUMN WILL SUSTAIN INDEFINITELY

1. *Introduction.* The maximum load which a reinforced concrete column can sustain indefinitely has long been of interest to designers. Series 4 of the column investigation was therefore designed for the purpose of ascertaining how large a load could be carried for a reasonably long time. The program of tests is given in Table 1.

TABLE 1—DESIGN STRENGTH OF CONCRETE, P. S. I.

L = longitudinal reinforcement, per cent
S = spiral reinforcement, per cent

Test Load Per Cent	Number of Columns Having Reinforcement of				Total No. of Columns
	4L + 0S Group A	4L + 1.2S Group B	4L + 2.0S Group C	6L + 2.0S Group D	
100	3	3	3	3	12
95	1	1	1	1	4
90	1	1	1	1	4
80	1	1	1	1	4
70	1	1	1	1	4
Total	7	7	7	7	28

The columns in this series of tests had an outside diameter of 8¼ in. and an overall length of 60 in. The concrete was designed for a strength of 3500 p. s. i. at the age of 56 days, and both the longitudinal and the spiral reinforcement were of intermediate grade steel. The cement and aggregates used were from the same supply as those used in the previously reported series. The method of making, storing and testing to failure of the columns was also the same as that used in other series.

Particular acknowledgment is made to C. L. Kreidler, formerly Research Fellow in Civil Engineering, for carrying out these tests and for the reduction of the data.

2. *Control Specimens.* The consistency of the concrete as measured by the slump cone, varied from 2½ to 4¼ in. for the four groups of columns. Due to a break-down of the 800,000-lb. testing machine Group B could not be tested at the scheduled age

*Presented at 29th Annual Convention, Chicago, Feb. 21-23, 1933.
†Research Assistant Professor of Engineering Materials, Lehigh University, Bethlehem, Penn.

of 56 days and was tested at an age of 112 days instead. The average compressive strengths of the concrete were 3780, 4140, 3430 and 3360 p. s. i. for Groups A, B, C and D respectively. The average results of the coupon tests of the reinforcement gave a tensile yield-point stress of 44,000 p. s. i. and an ultimate of 64,400 p. s. i. for the longitudinal steel used in the columns having 4 per cent reinforcement, and 44,700 and 70,000 p. s. i. for yield-point and ultimate stress for the columns having 6 per cent reinforcement. No yield-point stress could be obtained on the spirals. The ultimate strength of the spiral reinforcement was 85,500 and 74,700 p. s. i. for 1.2 and 2.0 per cent respectively.

To study whether the stressing of the longitudinal reinforcement beyond the yield-point stress in compression affected the load-carrying capacity of the reinforcement, compression tests were made on 3-in. long coupons which were cut from the ½-in. square bars used as reinforcement. The load-deformation curve for one of these coupons is shown in Fig. 1. It is noted that the load was released and reapplied at a strain below the yield-point, also at a strain slightly above the yield point, and again a a strain about fifteen times the initial yield-point strain of the steel. The bar continued to carry its full yield-point load at strains far above the yield-point strain. The yield-point stress was only slightly affected by excessive strains and the elastic properties of the steel remained the same. Although the strains were more than fifteen times the yield-point strain of the steel, the total load carried at these strains was only slightly greater than the original yield-point strength. The extreme amount of shortening shown in Fig. 1 is about 2.6 per cent.

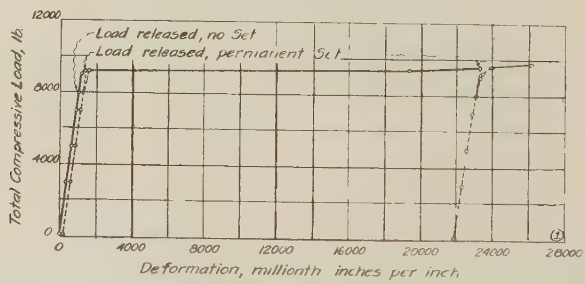

FIG. 1—LOAD-DEFORMATION DIAGRAM FOR ½-IN. SQ. BAR 3 IN. LONG CUT FROM LONGITUDINAL REINFORCEMENT ON STOCK PILE

3. *Columns.* Three columns of each group were tested "fast" directly to failure in the same manner as that described in the First Progress Report from Lehigh University.[1] A spherical bearing block was used at the top of the column and no attempt was made to restrict the motion of this block. The columns which were scheduled to sustain loads for a long period, were placed in a loading rig consisting of a number of heavy helical springs, three steel plates and four tension rods. (Shown in Fig. 9 in connection with illustrated test of Column 6.) A total of five rigs were made for these tests. The rods and plates were donated by the Bethlehem Steel Co. of Bethlehem, Penn., and 50 of the helical springs were loaned to the laboratory for three years through the courtesy of the Lehigh Valley Railroad Co.

The method of loading a column in the rig consisted of assembling the column and the loading rig on the floor of the laboratory and then placing the assembly on the table of the 800,000-lb. testing machine. The free space between the column and

the rods was ½ in. Load was applied by bringing the head of the machine down on the loading rig, thus compressing the column and the springs until the correct load was reached. At this stage there was no load in the tension rods. Two 10-in. gage-lengths had been placed on each rod. The gage lengths were on the opposite ends of the same diameter of the rod. Strain gage readings were taken on these gage lengths in order to establish a zero reading, the nuts were tightened on the rods, and the load exerted by the testing machine was released. The load on the columns was controlled by adjusting the nuts until the proper average strain was observed in the tension rods. Since the load was measured by the strains in the rods it was necessary to determine their modulus of elasticity. The resulting deformation diagram is shown in Fig. 2. The modulus of elasticity as taken from this diagram was 29,450,000 p. s. i.

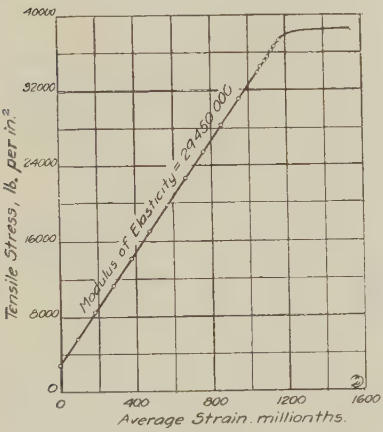

FIG. 2—STRESS-STRAIN CURVE FOR TENSION RODS USED IN LOADING RIGS

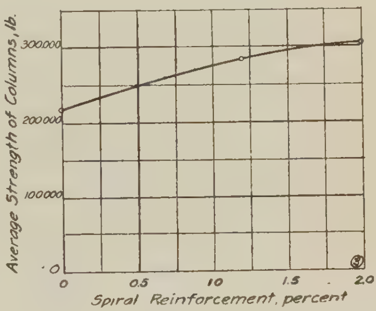

FIG. 3—EFFECT OF PERCENTAGE OF SPIRAL REINFORCEMENT ON ULTIMATE STRENGTH OF COLUMNS SAVING 4 PERCENT LONGITUDINAL REINFORCEMENT

The procedure for checking the load on the column was the same as for the loading. A new zero reading was taken on the rods each time the load was adjusted.

The summarized data of the column tests are given in Table 2. It is noted that 15½ months elapsed between the making of the first and the last columns of this series. The time of making the columns was governed by the time at which loading rigs were available. The deterioration of the cement as measured by the compressive strength, was small during this time.

4. *Fast Loading.* The results of the columns loaded "fast" to failure are presented in Table 2. In Fig. 3 the strengths of the columns having 4 per cent longitudinal reinforcement have been plotted against the percentage of spiral reinforcement. The effect of the amount of spiral reinforcement on the load-deformation curve is shown in Fig. 4. The three curves in Fig. 4 lie so close together that the modulus of elasticity of the concrete in the columns seems to determine the position of the curves. Since the modulus of elasticity varies with the strength of the concrete, the columns

[1]"First Progress Report on Column Tests at Lehigh University," by W. A. Slater and Inge Lyse. JOURNAL, American Concrete Institute, February 1931, *Proceedings*, Vol. 27, p. 677.

having the highest cylinder strength should show the least strain at a given load. The curves in Fig. 4 arrange themselves in order of their cylinder strength so that it is reasonable to conclude that the spiral reinforcement had no effect on the stress-strain relation within the range of loads for which strain measurements were taken.

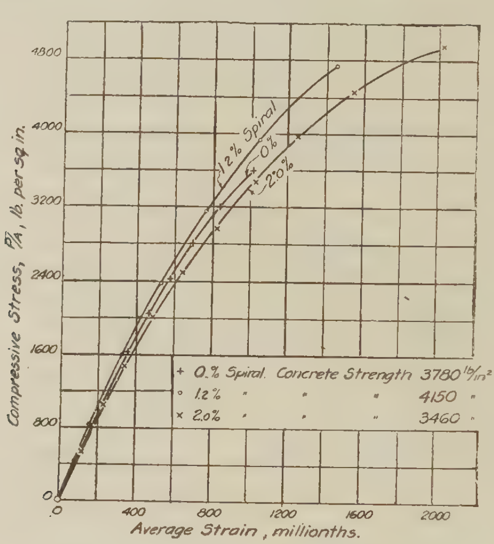

FIG. 4—STRESS-STRAIN DIAGRAMS FOR COLUMNS LOADED "FAST" TO FAILURE SHOWING EFFECT OF SPIRAL REINFORCEMENT ON THE STRAINES —LONGITUDINAL REINFORCEMENT 4 PER CENT

The effect of the amount of longitudinal reinforcement on the deformation of the columns is well illustrated in Fig. 5. It is noted that when the load carried by the longitudinal reinforcement is subtracted from the total load on the column, the stress-strain curves for the columns having 6 per cent and 4 per cent longitudinal reinforcement very nearly coincide. This means that the longitudinal reinforcement added its full stress value at any strain in the column.

5. *Time Effect.* In Group A the three columns loaded "fast" to failure gave an average ultimate load of 217,000 lb., which was taken as the 100 per cent strength of all the columns in this group. The columns in this group had 4 per cent longitudinal and 0 per cent spiral reinforcement. Column 4 was to be subjected to 95 per cent of the ultimate strength, but failed at a load of 93 per cent. Column 5 sustained 90 per cent of the load but held the load for only a few minutes. Column 6 was subjected to a load of 80 per cent, and Column 7 to 70 per cent. Because Column 6 showed no sign of distress it was deemed profitable to remove Column 7 from the rig and load it to failure, thus making the rig available for another column. Column 7 had been under 70 per cent of its ultimate load for 115 days when it was removed from the rig and loaded "fast" to failure. The deformation curves for Columns 6 and 7 are shown in Fig. 6. Column 7 developed lateral cracks during the release of the load. These cracks closed during the loading to failure and the column carried an ultimate load of 253,000 lb. or 17 per cent greater than the average ultimate load for the three

columns which were loaded to failure at the age of 56 days. The fact that the longitudinal reinforcement had been stressed far beyond its yield point during the time under load evidently did not have any detrimental effect upon the ultimate load of the column. Column 6, which sustained a load of 80 per cent of its ultimate, remained under this load for 700 days. The deformation curve for this period is shown in Fig. 7. It is noted that for the first year under load the deformation increased rapidly. From then on the deformation increased very slowly but did not stop entirely. The total deformation of the column was approximately four times the yield-point strain of the longitudinal reinforcement. Except for a few vertical cracks near the ends of the column, which developed shortly after the column had been placed under load, no sign of distress was present. It may be concluded, therefore, that this column would carry 80 per cent of its ultimate strength indefinitely.

TABLE 2—SUMMARIZED DATA ON COLUMNS

Col. No.	Date Made	Slump	Load		Remarks
		in.	%	lb.	
Group A—4% Longitudinal Reinforcement, 0% Spiral Reinforcement					
1	11-18-30	2½	100	220,000	Loaded "fast" to failure
2	11-18-30	3	100	202,500	Loaded "fast" to failure
3	11-18-30	2¾	100	229,000	Loaded "fast" to failure
4	11-19-30	2	95	206,000	Reached only 202,300 lb.
5	11-19-30	2½	90	195,000	Held load for only a few minutes
6	11-24-30	2½	80	174,000	Under test
7	11-24-30	2½	70	152,000	Held load for 115 days, loaded to failure, ult. load 253,000 lb.
Group B—4% Longitudinal Reinforcement, 1.2% Spiral Reinforcement					
8	1-21-31	3	100	282,000	Loaded "fast" to failure
9	1-21-31	5	100	290,500	Loaded "fast" to failure
10	1-21-31	6	100	275,000	Loaded "fast" to failure
11	1-22-31	5	95	268,000	Held load for 45 minutes
12	1-22-31	5	90	254,000	Held load for 65 hours, ult. load 286,800 lb.
13	1-27-31	3	90	254,000	Under test
14	1-27-31	3	—	—	Stored in laboratory under no load
Group C—4% Longitudinal Reinforcement, 2.0% Spiral Reinforcement					
15	12-10-31	3½	100	290,000	Loaded "fast" to failure
16	12-10-31	3	100	301,000	Loaded "fast" to failure
17	12-10-31	3	100	322,000	Loaded "fast" to failure
18	12-10-31	3	95	289,000	Held load for one day
19	12-22-31	2½	90	274,000	Held load for one day
20	12-22-31	4	85	258,000	Under test
21	12-22-31	3	80	243,000	Under test
Group D—6% Longitudinal Reinforcement, 2.0% Spiral Reinforcement					
22	1-28-32	3½	100	337,000	Loaded "fast" to failure
23	1-28-32	3½	100	355,000	Loaded "fast" to failure
24	1-28-32	3½	100	311,000	Loaded "fast" to failure
25	1-28-32	3½	90	302,000	Under test
26	3- 1-32	2½	—	—	Stored in
27	3- 1-32	3	—	—	moist room
28	3- 1-32	3	—	—	for later test

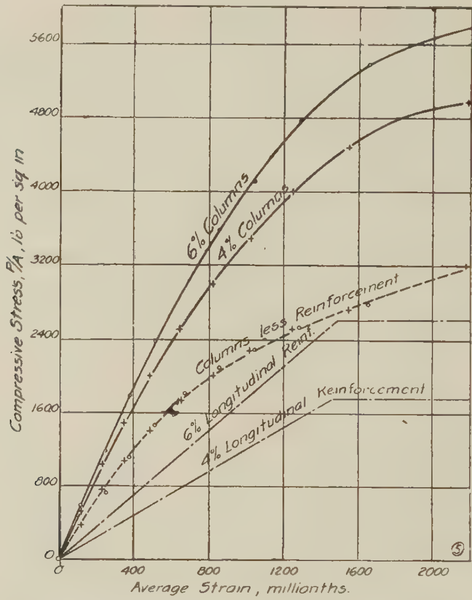

FIG. 5—STRESS-STRAIN DIAGRAMS FOR COLUMNS LOADED "FAST" TO FAILURE, LONGITUDINAL REINFORCEMENT 4 AND 6 PER CENT; SPIRAL 2 PER CENT

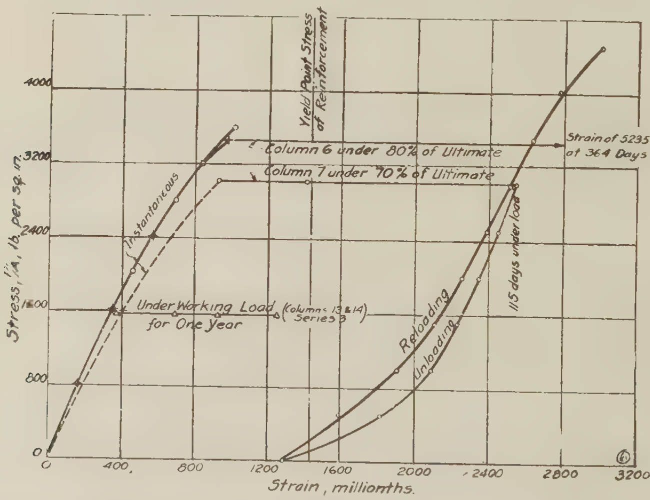

FIG. 6—STRESS-STRAIN DIAGRAM OF COLUMNS LOADED DIRECTLY TO FAILURE UNDER 80 PER CENT AND 70 PER CENT OF ULTIMATE LOAD. STRENGTH OF CONCRETE 3780 P. S. I. 4 PER CENT LONGITUDINAL AND 0 PER CENT LATERAL REINFORCEMENT

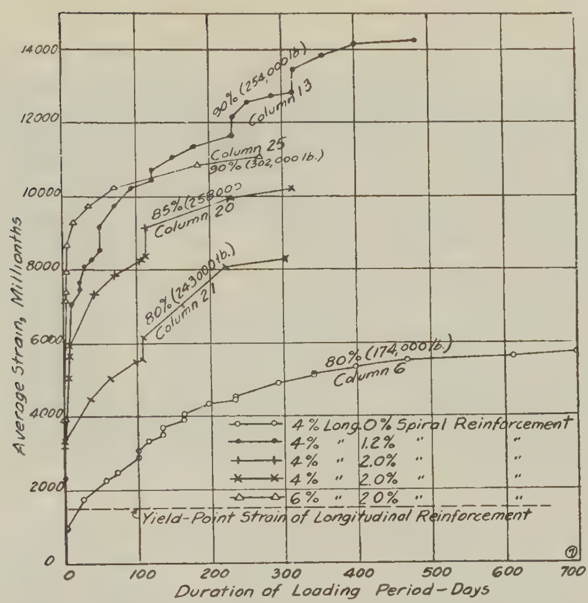

FIG. 7—DEFORMATION OF COLUMN UNDER SUSTAINED LOAD

As previously stated, the columns in Group B were tested at the age of 112 days. The columns in this group had 4 per cent longitudinal and 1.2 per cent spiral reinforcement. The average ultimate strength of the three columns loaded "fast" to failure was 282,000 lb. Column 11 was placed under 95 per cent of this ultimate load and held this load for 45 minutes before failing. Column 12 sustained 90 per cent of the ultimate for 65 hours but deflected laterally so much that it rested against the tension rods of the rig. It was therefore removed from the rig and loaded to failure. The ultimate load was 287,000 lb. Column 13 was also subjected to 90 per cent of the ultimate and has sustained this load for 500 days.* The deformation curve for this column is shown in Fig. 7, and it is noted that the strain is now approximately ten times the yield-point strain of the longitudinal reinforcement. Column 13 does not appear to be in immediate danger of failure, although the concrete outside the spiral has spalled off at several places as shown in Fig. 8. Column 14 was not placed under load but was stored with Column 13 as a control column for temperature and shrinkage strain. The maximum temperature and shrinkage strains to date correspond to a stress of approximately 7500 p. s. i. in the longitudinal reinforcement.

The columns in Group C had 4 per cent longitudinal and 2.0 per cent spiral reinforcement. The average ultimate load for the three columns loaded "fast" to failure was 304,000 lb. Column 18 held 95 per cent of its ultimate for one day, but had then buckled so badly that the test was discontinued. Column 19 held 90 per cent of its ultimate for one day. It had then buckled so much that it touched the tension rods and the test was discontinued. Column 20 was placed under 85 per cent of its ultimate and has been under this load for more than 300 days. The average strain in the

*On January 24, 1933.

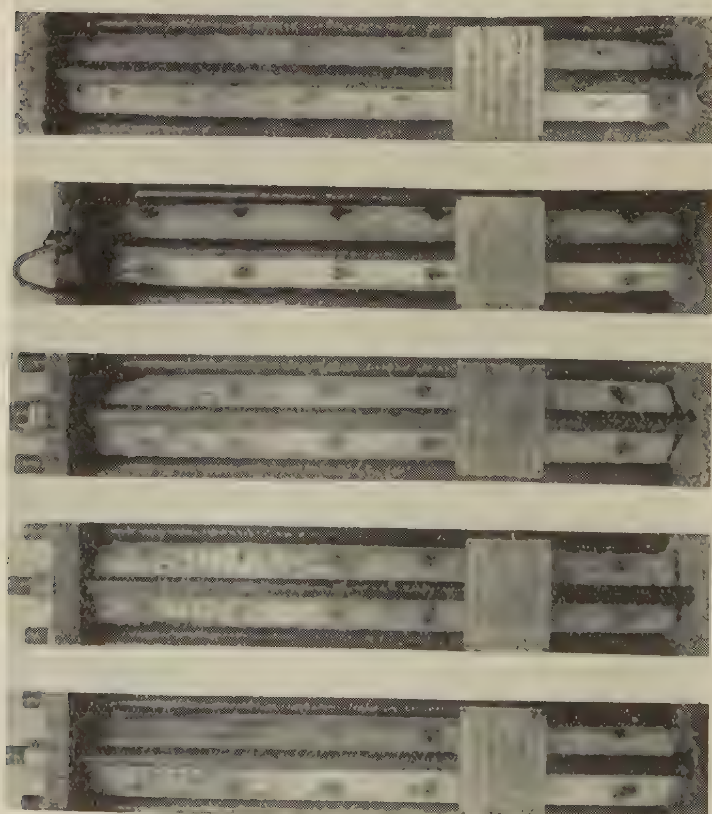

FIG. 8—APPEARANCE OF COLUMNS 6, 13, 20, 24 AND 25 IN JANUARY, 1933

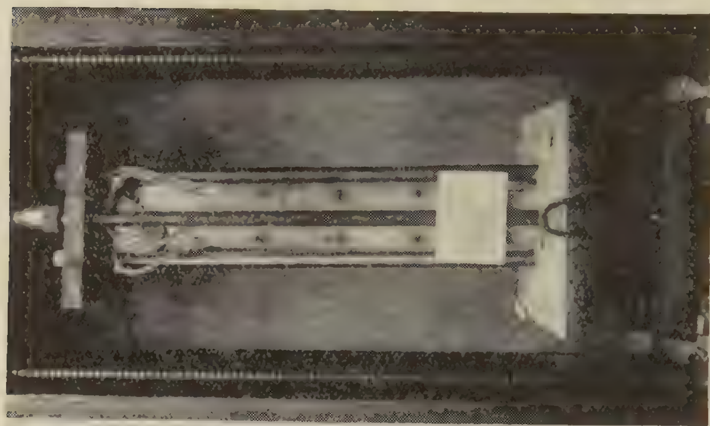

FIG. 9—FAILURE OF COLUMN 6. THIS PICTURE ALSO SHOWS THE LOADING RIG

longitudinal reinforcement is at present about seven times the yield-point strain of the steel. The column has gradually buckled under this load so that at present it is touching one of the tension rods of the rig. This column should therefore be considered as failing under 85 per cent of its ultimate load. Column 21 was placed under 80 per cent of its ultimate load and has sustained this load for more than 300 days. At present the average strain in the steel in this column is 5½ times its yield-point strain. The column shows no sign of immediate danger of failure, but it has buckled so much that it nearly touches the tension rods, and the concrete outside the spiral has begun to spall.

The columns in Group D had 6 per cent longitudinal and 2 per cent spiral reinforcement. The average ultimate strength of the three columns loaded "fast" to failure was 335,000 lb. Column 25 was placed under a load of 90 per cent of the ultimate and has sustained this load for nearly 300 days. The column deformed very much under this load and at present the average strain in the steel is about 7½ times its yield-point strain. The large deformation caused the concrete outside the spiral to spall at different places on the column and the column deflected so much that it is now touching one of the tension rods. Column 25 may therefore be considered as failing under a load of 90 per cent of its ultimate. Columns 26, 27 and 28 are being stored in the moist room for later tests, no loading rig being available as yet.

The time-strain curves for all the columns sustaining loads are shown in Fig. 7, and a photograph of these columns is presented in Fig. 8.

6. *Effect of Release of Load.* To subject the five loaded columns to a more severe condition than the mere sustaining of load, it was decided to release entirely and then reapply the load. An extra load of 10,000 lb. was placed on the column, the nuts on the tension rods loosened, and the load released. After a few minutes the load was reapplied and the column subjected to further testing. Column 6 was tested first. When the load was released the vertical cracks in the column extended further, but no other change was apparent. During the reapplication of the load the column showed no further sign of distress until the correct load of 174,000 lb. was reached. At this load the column failed with a loud report. The vertical reinforcing bars all buckled perpendicularly to the surface of the column as shown in Fig. 9. Evidently the release of the load caused the reinforcement to slip with respect to the concrete near the ends of the column. This slip caused extension of the vertical cracks, and when the load was reapplied the reinforcing bars were free to take on initial buckling. The bars were covered with only a ½ in. layer of concrete and therefore had little lateral restraint. Consequently the bars buckled.

Column 13 scaled to a larger extent during the reapplication of the load and also showed sign of initial bending. Otherwise no sign of distress appeared. The column is therefore sustaining a load of 254,000 lb. for an additional period.

Column 20 had buckled sufficiently to lean against the tension rods before the load was released. The column was moved away from the tension rods before the reapplication of the load. During the reapplication of the load the buckling increased to such an extent that the column touched the rods. Since it was deemed of little value to continue the loading of a column which was restrained against further buckling, the column was removed from the loading rig and loaded to failure. The maximum load was 385,000 lb., or 27 per cent more than the original strength of the companion columns.

Column 21 was subjected to the same cycle of test. Except for a slight extension of the flaking, the column showed no sign of increased distress. The column is therefore sustaining load for further tests.

Column 25 leaned heavily against the tension rods prior to the release of the load. As it had already been moved away from the rods the total buckling was about one inch, and the column was removed from the loading rig and loaded directly to failure. The maximum load was 417,400 lb., or 25 per cent more than the original strength of the companion columns.

The release of the load on these columns which had strains in the longitudinal reinforcement far above the yield-point strain of the steel, evidently produced additional distress in the columns. For the tied column which had only a ½-in. layer of concrete for protection of the reinforcement, the release of the load caused sufficient reduction in the lateral restraint to permit buckling of the bars.

7. *Summary.* While the number of tests included in this series was entirely inadequate for drawing final conclusions, the tests indicated that:

1. The stress-strain curve for columns loaded "fast" was substantially equal to the summation of the stress-strain curves for the longitudinal reinforcement and the concrete.

2. The amount of spiral reinforcement did not affect the stress-strain relation for the column at strains less than the yield-point strain of the longitudinal reinforcement.

3. The longitudinal reinforcement will carry its full yield-point stress at strains far beyond the yield-point strains.

4. The strength of the column was not decreased by being strained far beyond the yield point of its steel before the loading to failure.

5. A reinforced concrete column (tied or spiral column) will probably carry nearly 80 per cent of its ultimate load for an indefinite time.

6. A column of no spiral or a small amount of spiral reinforcement will carry 80 per cent of its ultimate load at less deformation and less sign of distress than will a column having a larger amount of spiral reinforcement.

7. The release of the sustained load caused additional distress in the columns and the tied column failed due to buckling of the reinforcing bars.

Discussion of Reports of Committee 105:

"Reinforced Concrete Column Investigation"

Tentative Final Report of Committee 105

Obviously April 1 was too early a closure date for the discussion of the Tentative Final Report of Committee 105 and the minority report by two members of the committee published in this Journal *for February and presented at the 29th Annual Convention. Discussion, now closed, is too voluminous to appear in this issue, other than to record additions to and revisions of the tentative report. Discussion is therefore continued to the October 1933,* Journal, *Vol. 30.*—Editor

Additions and Revisions, F. E. Richart, Chairman: The following minor changes and additions to the majority report as printed in the February Journal, were made on the Convention Floor. With these changes the report may be considered the FINAL REPORT OF COMMITTEE 105, and as such has been approved by those members who approved the printed Majority Report.

(1) Add to recommended provisions for spiral columns the following sentence: "The shell thickness outside the spiral shall not be less than 1½ inches, nor less than 1⅓ times the maximum size of the coarse aggregate, nor shall it be less than required by fire protection provisions under which the column may be built."

(2) Add to recommended provisions for tied columns the following sentence: "The lateral reinforcement in tied columns shall be at least ¼ in. in diameter, spaced not more than 12 in. apart."

(3) The maximum percentage of longitudinal steel in tied columns shall be 4 (instead of 3 as given in the printed report).

(4) Add the following provision as to slenderness ratio of columns: "The length of either tied or spirally reinforced columns to which the recommended design formulas of this report apply shall not exceed eleven times the least lateral dimension of the column."

Discussions Closed and Publication Deferred Until October (Vol. 30)

"Concreting Problems—Chats Falls Power Development"

—Errata—

Table 2, of this paper, February Journal, *Proceedings*, page 252. This volume should give weights of sand and stone for 2 cu. yd. batch as (table columns 2 and 3) sand 1311 lb., stone 1950 lb., instead of 2622 and 3900.

Painting on Concrete Surfaces

Report Committee 407. Published September 1932.

The Mortar Voids Method of Designing Concrete Mixtures

By Mark Morris. Published September 1932.

The Freyssinet Method of Arch Construction Applied to the Rogue River Bridge in Oregon

By Albin L. Gemeny and C. B. McCullough. Published October 1932. Discussion continued from February.

Reinforced Concrete Column Investigation

Amendments published in this Journal make final the tentative final report of Committee 105 published in February. Because of these amendments, because of a fifth progress report from Lehigh University (published this month) and the several discussions now available, all discussion is deferred.

Abstracts

Materials

Admixtures

Addition of calcium hydrate to concrete. *Tonind. Ztg.* (Germany), June 13, 1932, V. 56, No. 48, p. 614-5.—Review of report by Grün, on investigations of effect of addition of calcium hydrate to portland cement concrete and blast furnace slag cement concrete points out small amounts increase plasticity and water impermeability without affecting strength unfavorably. Specimens of 1:3 and 1:7 mixtures were stored from 1 month to 2 years in water, ammonium sulfate and magnesium sulfate. Additions of lime or ground sand increase strength slightly in water storage; they decrease strength, especially tensile, in ammonium sulfate.—A. E. B.

Change of chemical resistivity of portland cement against water by adding various admixtures. Karl Tremmel. *Zement* (Germany), June 2, 1932, V. 21, No. 22, p. 319-20—Referring to article by Gundius-Assarson—cf. *Zement* (Germany), Feb. 4, 11, 1932, V. 21, No. 5, 6, p. 64-6, 77-80; Journal A. C. I., June, 1932, V. 3, No. 10, *Abstracts* p. 189—2 means are possible for effective protection of concrete against corrosion caused by leaching action of aggressive waters: (1) addition of chemically active substances which combine with liberated lime to form insoluble products (siliceous materials) and (2) decreasing interior surface of concrete and filling voids and capillary openings to produce a concrete of max. density.—A. E. B.

Determination of fatty anhydrides in cement. J. L. Heitzman and George Coventry. *Rock Products*, June 4, 1932, V. 35, No. 11, p. 36.—Use of integral water proofing compounds calls for rapid method for their determination in cement. Such compounds are chiefly stearates of calcium or ammonia and stearic acid. Method described is to dissolve 20 g. in hydrochloric acid and remove fats and oils by mixing solution with 75 cc. of sulphuric ether and separating in a separatory funnel. Ether is evaporated in a weighed platinum dish. After weighing, residue is treated with 15 cc. chloroform to extract the fats, and then evaporated and weighed.—E. S.

Aggregates

The effect of varying moisture content of aggregates upon consistency, yield, amount of cement per cu. yd. and strength of concrete. Th. Kristen. *Zement* (Germany), June 16, 1932, V. 21, No. 24, p. 353-6.—Determinations of compressive strength of concretes made with various aggregates having natural moisture contents from 0 to 8% revealed a decided increase in strength with 1% moisture; between 4 and 5% moisture strength drops below initial strength (moisture free basis). Preliminary tests in laboratory with dry aggregates are suggested.—A. E. B.

Methods of testing mineral aggregates and associated materials. Prepared by National Slag Association for the Committee on Correlation of Research in Mineral Aggregates of the Highway Research Board, National Research Council, April, 1931. 192 p.—Seven divisions tabulate symposium of test methods on coarse aggregates, fine aggregates, concrete, hydraulic cement, bituminous cement, bituminous emulsions and bituminous mixtures.—D. F. J.

An investigation of the gradation of stone sand for concrete. A. T. Goldbeck. *Crushed Stone Jl.* May-June, 1932, V. 8, No. 4, p. 20-23.—Since stone sand is angular in shape proper gradation is of considerable importance in connection with strength, workability and economy of concrete. Mortar-voids tests were made on 17 gradations of stone sand of fineness modulus of 2.00 to 3.33. From data of mortar-voids test, concrete was designed to have a constant modulus of rupture and consistency as measured by slump test, which resulted in wide range in cement contents and workabilities. Workability was based on ease of manipulation in opinion of two operators. One series of specimens had an average modulus of rupture of 875 and other 560 p.s. i. at 14 days. The finer the sand, the greater was cement content required to produce concrete of a required strength. When concrete mixtures are to be made with a small amount of cement, a relatively fine sand should be used, if

workable concrete is required. For rich mixtures, it is quite feasible, desirable and economical to use a much coarser gradation of sand.—J. E. G.

Testing aggregates and steel bars. D. B. RUSH. *Concrete*, June, 1932, V. 40, No. 6, p. 27-28.—Field study of aggregate considers size and grading and quality of individual particles. Fine aggregate must be clean, hard and well graded within set limits. Size having been determined, quantity of silt is found by the decantation method. Moisture content, bulking, and hardness are next determined. Samples are obtained by quartering method. Possible lack of uniformity in chemical and physical properties of bars is prevented by having a representative at the manufacturer's plant during rolling of order to inspect and test material carefully before shipment. Inspectors witness tensile and cold-bend tests made on specimens selected from various sizes of bars in each lot. General inspection is made for surface defects, section and weight. —C. B.

Refined aggregate products for Hoover dam concrete. *Eng. News Record*, June 2, 1932, V. 108, No. 22, p. 783-7.—Unusual plant is required for preparing aggregate for 4½ million cu. yd. of concrete for Hoover Dam. All material is in a deposit on Ariz. side 8 miles above dam, and must be removed before 1935 to permit storage of water in reservoir. A 5-yd. electric dragline operates continuously removing overburden and loading gravel into 30-cu. yd. railway cars. These are hauled to screening plant or to storage pile on Nev. side, average output being 200 cars per day. Main features of screening plant are 4 steel towers 60 ft. high with connecting trusses supporting conveyor belts. Besides sand, aggregate is screened to 4 sizes: ¼-¾, ¾-1½, 1½-3, and 3-9 in. Fine aggregate of any desired fineness modulus may be produced from pit sand. Either wet or dry material may be handled, at possible capacity of 1000 tons per hour.—G. M.

New England crushed stone operation kept up to date. *Rock Products*, April 23, 1932, V. 35, No. 8, p. 27-29.—West Rocksbury Trap Rock Co., near Boston, Mass., has continually modernized plant so product meets more rigid specifications for aggregate. Flow sheet shows plant modernized with vibrating screens and cone crushers.—E. S.

Constant change and improvement the rule, even for gravel plants. *Rock Products*, June 18, 1932, V. 35, No. 12, p. 20-22.—Ohio Gravel Co., Cincinnati, O., changed Miamiville plant to provide suitable aggregate for Kentucky concrete highways. Grading was better after installing vibrating screens, with saving of power. To provide dryer sand than could be produced by sand drags of plant, sand as produced is loaded into gondola cars and held until thoroughly drained, then dumped into track hopper and elevated to loading bins.—E. S.

Crushed stone cleaning and sizing plant rebuilt for vibrating screens throughout. *Rock Products*, June 18, 1932, V. 35, No. 12, p. 9-11.—Genessee Stone Products Corp., Batavia, N. Y., rebuilt its 2000-ton crushing and screening plant primarily because state of New York prefers washed stone for aggregate in concrete highways. Crushed material to be washed is drawn from any concrete silo by belt conveyors and sent to double deck vibrating screens, with ⅛ and ⅝-in. mesh cloth, equipped with washing jets.—E. S.

Production of aggregates from river gravels in the plains region. Part 1. JOHN H. RUCKMAN, *Rock Products*, April 9, 1932, V. 35, No. 7, p. 20-21.—Analysis of 4 river bed deposits in territory drained by western tributaries of Missouri R. are given, with discussion of requirements of present day concrete specifications. Crude methods are giving place to production of material to meet exacting specifications. Part 2. June 4, 1932, V. 35, No. 11, p. 14-18.—For equipment adapted to shallow rivers of Middle West, barges are standardized, drawing about 16-in. when fully loaded. Boats drawing 8-in. have been found very convenient. With present design wood barges are as cheap as steel, and weight is no more. It is possible to improve design so that barges of half present weight may be made of rustless steel.—E. S.

Gasoline engine powered dredge for very shallow water. *Rock Products*, May 7, 1932, V. 35, No. 9, p. 34-35.—Hawkeye Material Co., Iowa City, Iowa, has special designed shallow-draft dredge which carries crushing and screening plant, made possible by bringing 100-ft. timbers from Oregon to provide necessary rigidity. —E. S.

A comparison of costs of quarrying versus mining of stone. GEORGE A. MORRISON. *Rock Products*, June 4, 1932, V. 35. No. 11, p. 31.—Figures compared are taken from questionnaire sent out by Bureau of Mines in 1929 on quarrying costs,

and from figures given at 1932 meeting of Nat. Cr. Stone Assn. for costs of mining stone. These are about the same, \$0.49 to \$0.531 for quarrying and \$0.515 for mining. Where quarry costs are low, mining costs also will be low. Mining has such advantages as cleaner stone and no interruption from weather conditions, but present mining practice requires leaving 40 to 60% of deposit to support workings.—E. S.

Field conveyors from pit to preparation plant save money. *Rock Products*, May 21, 1932, V. 35, No. 10, p. 27-28.—In operation at Paris, Ontario, and near Los Angeles, Calif., long conveyors bring material from pit to washing and screening plant. At Canadian operation there was reduction of pit labor from 17 to 4 men, and handling capacity was increased 30%. In California operation costs were 5½¢ per ton with tracks, locomotives and hoists. With belt cost is 2½¢ per ton.—E. S.

Sand and gravel dry excavated pumped to top of plant. *Rock Products*, May 21, 1932, V. 35, No. 10, p. 11-13.—In operation of Gifford-Hill and Co's plant at Trout, Louisiana, material excavated by drag-line is pumped considerable distance from pit and 70 ft. to top of screening plant as cheapest means of conveying and elevating. It also aids in freeing material from clay balls.—E. S.

From shovel to dredge excavation after a year's operation. *Rock Products*, April 9, 1932, V. 35, No. 7, p. 17-19.—Bristol Sand and Gravel Co., Bristol, Penn., after a year's bank digging developed pit big enough to work suction dredge which operated very well with aid of traveling nozzle. Both pump house and frame for traveling nozzle rest on cylindrical pontoons.—E. S.

Huge blast at Inland quarry dislodges 1,250,000 tons of material. A. J. CAYIA. *Pit and Quarry*, V. 24, No. 3, May 4, 1932, p. 33-36, 38.—Details are given of largest commercial blast on record. After months of preparation by Inland Lime & Stone Co., 220 tons of explosives, loaded in 4,000 holes, shattered a year's supply of limestone to be consumed as flux in Inland Steel Co.'s works at Chicago.—A. J. H.

Large blast holes as a substitute for coyote holes in moderate size traprock quarry. *Rock Products*, May 7, 1932, V. 35, No. 9, p. 26-28.—Keystone Trap Rock Co., developed use of 12-in. well-drill holes in place of coyote holes. Hole was sprung with 40% powder, then loaded with 2 tons 80% gelatin followed by 2 tons 60% gelatine. Eighty thousand tons of stone was secured from one 12-in. hole and four 8-in. side holes, 3.08 tons per lb. explosive.—E. S.

The volume delusion. J. C. BUCKBEE. *Rock Products*, April 23, 1932, V. 35, No. 8, p. 38-39.—In discussing relation between output and cost, gravel plant is taken as example showing cases in which increase of business actually results in higher cost and lessened profit. In addition attempt to get business by cutting price disturbed market and started price-cutting war. Cost curve shows very considerable increases in volume of output do not reduce cost very much.—E. S.

CEMENT

New English standard specifications for portland cement. *Tonind. Ztg.* (Germany), May 2, 1932, V. 56, No. 36, p. 472-3.—New specifications include: definition of portland cement, strength requirements, setting time, volume consistency, chemical composition and fineness.—A. E. B.

German specifications for portland cement, iron-portland cement and blast furnace slag cement. 1932, Zementverlag, Charlottenburg, Germany, 46 p. Reviewed in *Tonind. Ztg.* (Germany), June 6, 1932, V. 56, No. 46, p. 593.—Publication presents official German standard specifications.—A. E. B.

Chemical analysis for standard cements. 1931, Zementverlag, Charlottenburg, Germany, 18 p. Reviewed in *Tonind. Ztg.* (Germany), June 6, 1932, V. 56, No. 46, p. 593.—Standard method for chemical analysis which was agreed upon after careful study by numerous German investigators is practical supplement to cement specifications (see preceeding abstract).—A. E. B.

Quick setting cements. NITZSCHE. *Tonind. Ztg.* (Germany), May 9, 1932, V. 56, No. 38, p. 499-500.—Setting properties of new type of cement for water tightening jobs in mines can be accelerated by certain admixtures without lowering strength very much. Cement must be stored very carefully. Mixing water to produce normal consistency should be about 2% more than for normal cements. Volume changes of specimens in water storage are very favorable while shrinkage in air is slightly greater than in case of normal cements. Absorption and evaporation of water from test slabs are very much like normal cement slabs.—A. E. B.

Contribution to the hardening problem of portland cement. K. KOYANAGI. *Zement* (Germany), May 5, 1932, V. 21, No. 18, p. 257-9.—Hardening phe-

nomina reported by Tippmann—cf. *Zement* (Germany), Dec. 25, 1930, V. 19, No. 52, p. 1225-34; Aug. 20, 1931, V. 20, No. 34, p. 774-81; JOURNAL A. C. I., May, Dec. 1931, V. 2, 3, No. 9, 4, *Abstracts* p. 248, 81—were studied by treating pure calcium carbonate, burned at 2012° F., with distilled water, saturated gypsum solution and gypsum solutions of various concentrations. Samples in distilled water showed under miscroscope very small hexagonal crystals. Lime and gypsum solutions produced well developed calcium hydrate crystals together with amorphous calcium hydrate. Solubility determinations of lime in water and gypsum solutions revealed a supersaturation in saturated gypsum solution during first minutes of reaction, then solubility drops below that of lime in distilled water. Fine needle crystals found by Tippmann and considered to be modification of calcium hydrate were calcium sulfoaluminate.—A. E. B.

Fine particle composition and quality of cements. DESIDER STEINER. *Zement* (Germany), June 23, 1932, V. 21, No. 25, p. 367.—Continuation of polemic between author and Weissgerber—cf. *Zement* (Germany), Apr. 21, 28, June 9, 1932, V. 21, No. 16, 17, 23—concerning effect of fine grinding upon strength of cement and theory derived by Weissgerber.—A. E. B.

Grinding of fine cement. A. EIGER. *Zement* (Germany), June 16, 1932, V. 21, No. 24, p. 348-9.—Highest strength is not obtained with cements with great percentage of very fine grains but by increasing amount of medium size particles (10 to 40 microns). Such suitable gradation is obtained by grinding clinker in a 2-compartment compound mill with specified charge of steel balls.—A. E. B.

Fine grained cement. ANTONIE EIGER. *Tonind. Ztg.* (Germany), May 23, 30, 1932, V. 56, No. 42, 44, p. 532-3, 558-60.—Increase of interior surface produces rise in compressive strength only to certain limit with approximate proportionality. Development of tensile strength seems to follow Feret's formula. Max. does not become noticeable when comparison is made upon basis of same water addition. Strength ceases to increase when interior surface is greater than 620 sq. in. per gram. Best strength is obtained when entire cement goes in solution during hydration process. Cement should contain as much as possible of particles from 10 to 40 microns and as little as possible of particles between 0 and 10 microns. Method is pointed out to determine relations between hydration and strength of cements by observing reaction curves and particle size distribution.—A. E. B.

Notes on the determination of flour in cement. EDUARDO TAYLOR. *Rock Products*, April 9, 1932, V. 35, No. 7, p. 46-47.—Flour, the active and strength producing portion of cement, is defined as those particles with diams. between 10 and 25 microns. Simple sedimenting devise for finding this fraction is cylinder filled to definite mark with absolute alcohol. Sample of cement is added and mixture thoroughly agitated by air jet. Mixture then is allowed to settle for a definite time and the alcohol is siphoned down to a mark on cylinder. This is filtered and wt. of cement carried by it is determined.—E. S.

Keeping constant boiler water level in test of cement. C. H. LOVEJOY. *Rock Products*, June 4, 1932, V. 35, No. 11. p. 37.—Device is an adaption of the ordinary five-gallon bottle drinking fountain used in offices. As water in boiler used for soundness test on cement boils away it is replaced from bottle by reason of a difference in level.—E. S.

Effects of storage on strength of cement. KATSUZO KOYANAGI. *Rock Products*, May 21, 1932, V. 35, No. 10, p. 38-39.—In one series of recent tests of storing cement samples were spread on paper on laboratory tables and in another cement was stored in hemp bags in laboratory. Five of the 6 cements tested showed increased strength after one month. This cement was low in lime. Differences in lime content might account for differences in results previously obtained.—E. S.

Volume delusion in the cement industry. J. C. BUCKBEE. *Rock Products*, May 7, 1932, V. 35, No. 9, p. 30-32.—Calculations and graphs show that lowering of costs by increase of volume is not so great as it is usually supposed to be. With 40% fixed charges costs are lowered only 7% by increasing the volume 20%. When times are good lower cost from increased volume sometimes does not follow because small economies are not practiced as when business is poor. Excess capacity is not cause of present ruinous reduction of prices, because there was much so-called excess capacity during peak year of 1928.—E. S.

Economic delusion. C. J. BUCKBEE. *Rock Products*, May 21, 1932, V. 35, No. 10, p. 47.—While we are bound by tradition to make concessions in prices when business is slow, goal should be steady prices. This may more nearly come about

when our executives rid themselves of volume delusion and refuse to sell at less than cost and adopt live-and-let-live policy.—E. S.

Produce various colors of clinker with treated raw materials. *Concrete*, V. 40, No. 7, p. 36—U. S. patent No. 1,829,082, granted Oct. 27, 1931, covers a process of combining coloring matter with cement during burning. Phosphate, borates, halogen compounds, oxide of chromium, nickel, cobalt or copper are used. Pure green cement results from the mixing of 100 kg. of ground raw material, 4 kg. of fluor-spar, 1 kg. of borax, 0.5 kg. chromium oxide, burned at sintering temperature in an oxidizing flame.—C. B.

Dusting of portland cement clinker. ALTON J. BLANK. *Rock Products*, April 23, 1932, V. 35, No. 8, p. 44-45.—Author summarizes what is said by LeChatelier, Rankin, Nacken and Dyckerhoff, Erdahl and Koyanagi about dusting of cement clinker, giving chemical composition of dusting clinkers tested by these authorities. Number of factors may cause dusting and causes at one plant may be quite different from causes at another. Problem of dusting cannot be solved with laboratory experimental mixes.—E. S.

The French cement industry. M. UEBELHOER. *Zement* (Germany), June 16, 23, 1932, V. 21, No. 24, 25, p. 357-8, 373-4.—Review of present situation of French cement industry gives detailed description and production figures of larger cement plants and their mechanical installations.—A. E. B.

The cement plant in Settenz, Czecho Slovakia. *Tonind. Ztg.* (Germany), May 19, 1932, V. 56, No. 40-41, p. 525.—Brief description of recently built portland cement plant equipped with 2 high efficiency shaft kilns. —A. E. B.

Making cement in the Far East. H. M. POWER. *Rock Products*, May 7, 1932, V. 35, No. 9, p. 17-22.—Cebu Portland Cement Co., Cebu, P. I., originally built a plant for 1000 bbl. per day, but due to changes in equipment it is now producing 1400 bbl. daily by wet process. Details of operation are included.—E. S.

The large packing installations of the portland cement plant in Origny Ste. Benoite, France. CARL HEROLD. *Tonind, Ztg.* (Germany), May 26, 1932, V. 56, No. 43, p. 548-9.—French cement plant is equipped with modern and economically operating packing equipment: 10 packing machines of Haver system for valve bags. Four different brands of cement can be packed and loaded simultaneously either in freight cars or boats.—A. E. B.

New cement mill on Sonora River serves Mexico's west coast territory. *Pit and Quarry*, Apr. 20, 1932, V. 24, No. 2, p. 14-16.—Limestone, shale and clay occur abundantly contiguous to this wet-process plant of 500-bbl. capacity. Kiln fuel is oil. All power is furnished by Diesel-driven generators.—A. J. H.

Super-cement plant designed to yield a quality product at lower cost. AMAN MOORE. *Pit and Quarry*, Apr. 20, 1932, V. 24, No. 2, p. 23-26, 32.—Ground plans and complete flowsheet are given for plant to produce quick-hardening portland cement, white cement, marine cement, gypsum products, magnesite products, fertilizer products, raw and burned lime products, with gas-ice as by-product.—A. J. H.

Automatic bag filters collect kiln dust at Ford Motor Co. cement plant. E. H. DECONINGH. *Pit and Quarry*, V. 24, No. 2, Apr. 20, 1932, p. 27-28, 41.—In Dearborn, Mich., mill kiln exhaust passes through a waste-heat boiler, economizer, fan, heat exchanger, bag filters, another fan, to stack. Bags are automatically cleaned frequently.—A. J. H.

Low installation and operating costs features of dust collector. *Concrete*, June, 1932, V. 40, No. 6, p. 51-53.—Dust collecting system at LaSalle, Ill., plant of the Marquette Cement Mfg. Co. is hooked up with the waste heat boiler system that utilizes heat of kiln gases. Unit constitutes closed system, so designed that dust-laden air or gases may either be blown through or drawn through.—C. B.

Sintering grate and rotary kiln gain fuel economy. T. H. ARNOLD. *Concrete*, June, 1932, V. 40, No. 6, p. 45-50.—In tabular and graphic comparisons of fuel and power costs of 6 different burning processes, considerable saving is shown in favor of Lepol process. Typical plants chosen were as follows: Wet process with comparatively short kilns, filters for dewatering slurry, and waste heat boilers to reclaim heat in waste gases; dry process with comparatively short kilns, and waste heat boilers; wet process with long kilns, equipped with chains, using purchased power and operated to obtain max. output; wet process with same conditions as previosuly mentioned, except operation is for max. economy; dry process with long kilns for efficient heat transfer, and purchased power; dry process with short kilns, sintering grate and purchased power.—C. B.

Electric dewatering of slurry under four-year test. HEWITT WILSON AND H. G. WILCOX. *Concrete*, June, 1932, V. 40, No. 6, p. 55-57.—Facts shown by laboratory experiments on electrophoresis with cement slurry tested include: Limestone and clay both carry negative charge and are deposited without apparent separation on anode. Cement slurry can be dewatered to some extent without addition of an electrolyte, but proper electrolytes will increase fluidity, rate of recovery, and dryness of deposit. Increasing cathode area by use of laminated cathode does not affect results.—C. B.

The manufacture of portland cement. Part 2. S. E. HUTTON. *Rock Products*, April 23, 1932, V. 35, No. 8, p. 32-33.—Raw materials are considered and manufacturing process classified into 12 fundamental operations. Costs are classified as manufacture, delivery, selling and overhead. Discussion of manufacturing costs begins with value of raw materials. Part 3. May 21, 1932, V. 35, No. 10, p. 19-21.—Author discusses in general terms handling, storing and processing materials, controlling and delivering and selling the product. Part 4. June 18, 1932, V. 35, No. 12, p. 13-15.—Present method of thinking of ratios will soon be superseded by dealing with percentages of definite chemical constituents found in cement. Table of typical cement analyses shows range of 60-67% CaO and of 20-24% SiO_2. Significant ratios are useful for superficial consideration of raw materials. Formation of 4 principal compounds of lime with alumina, silica and iron is discussed with equations. A final formula is given but from intricate interelations of raw mix components and cement constituents such formulas are more or less irrational and may be innumerable.—E. S.

Grinding plant research. Part 4. WILLIAM GILBERT. *Rock Products*, April 23, 1932, V. 35, No. 8, p. 40-41.—Machines used in coal grinding were kominor and tube mill in series. Coal was slack with 19.5% ash and 2.2% moisture. Kominor was found to be more efficient than tube mill, in ratio 100:83.5. Test also was made with same arrangement working on coal with 9.3% ash dried to 1.92% moisture. Efficiency of flint stone chamber was highest. Both balls and flints used were too large. —E. S.

Improved grinding methods in the Lehigh Valley district. EARL C. HARSH. *Rock Products*, May 21, 1932, V. 35, No. 10, p. 31-33.—Whitehall Cement Manufacturing Co., Cementon, Penn., has installed a new pulverizing mill working on principle of ball bearing. Feed from hammer mills, 1½-in. and under, has been reduced to 90% through 200 mesh at rate of 35 tons per hr., power consumption being 9.3 k. w. hr. per ton. Mill is in closed circuit with a 16-ft. air separator. Operation is practically dustless.—E. S.

The drying and grinding operation in the cement industry. WILHELM JAEDEL. *Tonind. Ztg.* (Germany), May 23, 1932, V. 56, No. 42, p. 536.—Referring to previous publications—cf. *Tonind. Ztg.* (Germany), Oct. 12, 1931, Jan. 1, 1932, V. 55, 56, No. 82, 1, p. 1147-9, 6-7—author compares efficiency of installations which dry and pulverize raw mixture in one operation. Effect of size of steel balls, addition of coke to raw mixtures and degree of fineness are pointed out.—A. E. B.

Construction of suction filter. *Rock Products*, May 7, 1932, V. 35, No. 9, p. 48-49.—A rapid filter for cement chemists uses tantalum cone in glass funnel, stem of which passes through cork in top of bell jar. Side opening in bell jar admits pipe to vacuum pump. Important feature is closing of bottom of bell jar without leakage. It sets on a gasket drawn down tight to bench with steel disk and bolt and nut.—E. S.

Studies on hydrothermal synthesis of calcium silicates under ordinary pressure Part 3. SHOICHIRO NAGAI. *Kogyo Kwagaku Zasshi, Journal of Soc. Chem. Ind.* (Japan), April, 1932, V. 35, Supplemental binding No. 4, p. 153-6B.—Paper continues previous comprehensive studies—cf. *Kogyo Kwagaku Zasshi, Journal of Soc. Chem. Ind.* (Japan), Oct., Nov. 1931, V. 34, Supplemental binding No. 10, 11, p. 378-81B, 418-22B; JOURNAL A. C. I., Feb. 1932, V. 3, No. 6, *Abstracts* p. 129—and describes comparative studies of calcium-silica mixtures ranging from 1:1 to 3:2 heated for long time periods at 1472° F. in water vapor of atmospheric pressure. —A. E. B.

Studies on synthesis of calcium silicates. Part 2. SHOICHIRO NAGAI AND KEI-ICHI AKIYAMA. *Kogyo Kwagaku Zasshi, Journal of Soc. Chem. Ind.*, (Japan), Jan., 1932, V. 35, Supplemental binding No. 1, p. 8-10B.—Continuing previous studies —cf. *Kogyo Kwagaku Zasshi, Journal of Soc. Chem. Ind.* (Japan), Dec. 1931, V. 34, Supplemental binding No. 12, p. 471-3B; JOURNAL A. C. I., April 1932, V. 3, No. 8,

Abstracts p. 159—author reports results of synthetic preparation (thermal synthesis) of calcium silicates and characterizes their hydraulic properties. Dicalcium silicate was prepared and its strength tested by small-piece testing method designed by Nagai. Part 3. *Ibid*, Feb. 1932, V. 35, Supplemental binding No. 2, p. 65-7B.—Tricalcium silicate was prepared from various mixtures including such containing ferric oxide. Part 4. *Ibid*, March 1932, V. 35, Supplemental binding No. 3, p. 118-22B.—Paper describes intensive studies of ternary system CaO-SiO_2-Al_2O_3 and of quarternary system CaO-SiO_2-Al_2O_3-Fe_2O_3.—A. E. B.

High alumina cement with high iron content. V. RODT. *Zement* (Germany), June 16, 1932, V. 21, No. 24, p. 352-3.—French patents No. 571,572 and 587,343 describe process for more economic manufacture of high alumina cement in which part of alumina is replaced by ferric oxide. Produced cement is sound and highly resistant against aggressive solutions.—A. E. B.

The heat calculation of cement kilns. EHRHARD SCHOTT. *Zement* (Germany) June 2, 16, 1932, V. 21, No. 22, 24, p. 315-9, 349-52.—Author investigates heat distribution in various kinds of cement kilns and presents heat flow diagrams for rotary and shaft kilns. Theoretical amount of heat for formation of portland cement clinker was determined with detailed description of heating of raw materials, calcination, clinkering, exothermic heat development and recovery of heat from clinker. Method enables comparative valuation of different types of kilns.—A. E. B.

Fundamental study of the heat technique in burning cement. WILHELM EITEL AND ERNST SCHWIETE. *Zement* (Germany), June 23, 1932, V. 21, No. 25, p. 361-7.—Author illustrates thermo-chemical phenomena of cement clinker burning by first deriving theoretical foundations and then studying their application for cement mill practice. Amounts of heat were determined experimentally by reproducing various steps of clinker burning process. Heat flow diagrams for an ideal iron-free and for a commercial clinker are presented.—A. E. B.

The raw material mixture for high quality portland cements. TH. KLEHE. *Tonind. Ztg.* (*Germany*), June 6, 1932, V. 56, No. 46, p. 587-8.—Author characterizes practical value of best composition formulas based upon silica modulus and hydraulic modulus and established by theoretical deductions. Magnesia does not enter into calculation. Silica modulus must be corrected for ferric oxide content in clinker. Ideal cement with silica modulus of 2.7 and hydraulic modulus of 2.4 should contain 20.56% SiO_2, 5.90% Al_2O_3, 2.88% Fe_2O_3, 67.66% CaO, 2.00% MgO and 1.00% minor constituents. Kuehl's formula is briefly discussed.—cf. *Tonind. Ztg.* (Germany), Jan. 28, March 7, 1932, V. 56, No. 9, 20, p. 118-20, 275-7; JOURNAL A. C. I. May 1932, V. 3, No. 9, *Abstracts* p. 174, 176.—A. E. B.

Flue dust deposits of rotary kiln installations. D. STEINER. *Zement* (Germany), May 5, 1932, V. 21, No. 18, p. 255-7.—In systematic investigation of various dust deposits from rotary cement kilns equipped with waste heat boilers and Oski dust collecting systems, chemical analyses of samples taken at various points of system enabled calculation of different compounds in dust: coal, coal ash, raw mixture, clinker, alkali and sulfur.—A. E. B.

MISCELLANEOUS

Rapid method for determination of silica. *Tonind. Ztg.* (Germany), June 9, 1932, V. 56, No. 47, p. 597-8.—Two methods are generally used to facilitate filtration of fine dispersed silica: (1) application of special membrane filters which necessitate use of special apparatus and (2) production of coarse silica by dehydration with certain acids. Use of concentrated sulfuric acid or perchloric acid is suggested. Very small quantities may be determined by colorimetric method.—A. E. B.

Grinding plant research. Part 6. WILLIAM GILBERT. *Rock Products*, June 4, 1932, V. 35, No. 11, p. 24-27.—Coal was ground in kominor and tube mill, also in compound tube mill. In first test very high efficicney was secured, thought to be due to use of lifter bars on liners, but equally good results were secured with smooth liners. Power consumption was 27.8 h.p. With compound tube mill power consumption was 37.8 h. p.—E. S.

PROPERTIES OF CONCRETE

Form pressure of pumped concrete. K. MAUTHNER. *Beton u. Eisen* (Germany), May 20, 1932, V. 31, No. 10, p. 160-162.)—Pressure readings were taken at various points in all types of structural members, poured by means of concrete pump. As long as pumping continues and concrete mass is kept in motion in forms,

pressure is that of fluid. Cessation of this motion causes settlement and concrete assumes properties of semi-fluid. This continues until loss of water and setting process have exerted their influence.—A. A. B.

X-ray studies of concrete. C. KANTNER. *Beton u. Eisen* (Germany), June 5, 1932, V. 31, No. 11, p. 170-174.—Examination of reinforced concrete members by means of X-rays has shown possibility of determining amount and shape of reinforcement, density of concrete and presence of cavities or foreign bodies by this method. Five X-ray photographs of a concrete beam are shown to indicate the effectiveness of this manner of investigation.—A. A. B.

Practical methods for testing masonry mortar cements. F. O. ANDEREGG. *Rock Products*, June 4, 1932, V. 35, No. 11, p. 28-30.—Author gives details of tests, mostly new, developed at Mellon Institute, to determine workability, adhesion to masonry units, water-tightness, weather resistance, flexibility, shrinkage and shrinkage rate, compressive strength, efflorescence, fading and staining of cement mortars. Mortar color pigments should be added where they are to be used in practical work. —E. S.

The quality of concrete and the question of its strength. Study of 50-year old concrete samples taken from structures. LUZ DAVID. *Tonind. Ztg.* (Germany), May 30, 1932, V. 56, No. 44, p. 560-2.—Author illustrates briefly various relationships between concrete consistency, water cement ratio, aggregates and quality of cement. Compressive strength data and chemical analyses are given of concrete samples cut out of 50-yr.-old hardened masses.—A. E. B.

Experience with the use of concrete for boiler foundations. WILHELM STUEMER. *Bautenschutz* (Germany), Feb. 5, 1932, V. 3, No. 2, p. 23-4.—R. c. is suitable for boiler foundations; parts exposed to heat radiation should be lined with refractories. Supporting r. c. beams of travelling grate of one boiler never reached temperature above 320° F. and gave good service. Center beam of second boiler failed completely due to temperatures from 464 to 516° F.—A. E. B.

The action of carbon dioxide upon cement mortar. C. R. PLATZMANN. *Bautenschutz* (Germany), April 5, 1932, V. 3, No. 4, p. 44-6.—After discussing views of Passow, Natta, Fontana, Gonell, Goslich and Hart about chemical reactions between CO_2 and lime liberated from cement, author illustrates mechanical action of this gas which causes excessive shrinkage and expansion as well as heat development. —A. E. B.

New experiments with the chemical consolidation process by Joost. KLEINLOGEL. *Bautenschutz* (Germany), May 5, 1932, V. 3, No. 5, p. 49-54.—Laboratory experiemnts show that concrete specimens of different cement content and composition are attacked by aggressive solutions while similar specimens show an increase in strength and resistivity against aggressive waters when treated by chemical consolidation process either by direct treatment of masses and tightening of voids or after removal of harmful salts already formed in concrete. Specimens were stored in sodium sulfate solutions for periods of 6 and 12 mo.—cf. *Bautenschutz* (Germany), Jan. 5, 1931, V. 2, No. 1, p. 4-8; JOURNAL A. C. I., Dec. 1931, V. 3, No. 4, *Abstracts* p. 86.—A. E. B.

Testing of cements for oil wells. ALBERT HEISER. *Tonind. Ztg.* (Germany), June 6, 1932, V. 56, No. 46, p. 585-6.—Conditions in wells (high pressures and temperatures, aggressive waters) require cement pastes made with 50 and more % water. Setting time of paste (3-5 hr. initial; 10-15 hr. final set) is determined by immersing Vicat form with specimen into solution. Strength is generally determined as flexural strength with 0.8 by 0.8 by 15.7 in. prisms. Sedimentation of various grain sizes in forms takes place due to thin consistency. Mixtures show considerable loss in strength, especially during first few days. Additions of calcium chloride (1.5 to 3%) improved strength about 68%. Kuehl cement was found to be most suitable cement with lowest shrinkage.—A. E. B.

Hardening properties of cement. DESIDER STEINER. *Tonind. Ztg.* (Germany), Apr. 18, 1932, V. 56, No. 32, p. 428-9.—Author illutrates reactions between cement and water and stresses especially secondary development of strength in partly hydrated loose cement and in cement mortar specimens (regeneration of strength in broken test cubes or in concrete in structures). Compressive and tensile strength developments of cement stored under various conditions were studied. When water is driven off from set cement part of hardening energy is recovered.—A. E. B.

Contribution to the knowledge of high alumina cement. Behavior of mortar and concrete at high temperatures. H. VIERHELLER. *Tonind. Ztg.* (Germany), May 2, 1932, V. 56, No. 36, p. 470-1.—Observations by Haegermann—cf. *Tonind. Ztg.* (Germany), Dec. 1931, V. 55, No. 103, p. 1430—that several high alumina cements show appreciable retardation of setting time at temperatures of about 73° F. is of no direct importance for concrete production. Only experiments with specimens of large masses and small surfaces are comparable with practice. Development of heat in interior of concrete is of far greater importance than outside temperature. Experimental results were obtained with various cements and concretes tested at room and higher temperatures. No harmful effect upon strength development was noticed at latter temperatures.—A. E. B.

Studies on the action of sulphates on portland cement. 4. The action of sulphate solutions on mortars prepared from some binary and ternary compounds of lime, silica, alumina and iron. T. THORVALDSON, D. WOLOCHOW AND V. A. VIGFUSSON. *Can. Jl. Research*, May, 1932, V. 6, p. 485-517.—Study was made of the action of solutions of the sulphates of magnesium, sodium and calcium on 1:00 mortar prisms made with standard sand and following substances or mixtures of these: tricalcium silicate, β-dicalcium silicate, γ-dicalcium silicate, tricalcium aluminate, $5CaO \cdot 3Al_2O_3$ monocalcium aluminate, $3CaO \cdot 5Al_2O_3$, dicalcium ferrite, and $4CaO \cdot Al_2O_3 \cdot Fe_2O_3$. Some experiments dealt with mortars of richer mix (1:7½ and 1:5). Effect of solutions was determined by measuring linear expansion of prisms and tensile strength when measurements of expansion were discontinued. A very pronounced difference was found to exist between the behavior of mortars made with mixtures rich in tricalcium silicate and those rich in β-dicalcium silicate. This observation is applied in discussion of resistance of different types of hydraulic cements to action of sulphate solutions.—AUTHORS' ABSTRACT

ENGINEERING DESIGN

BUILDINGS

New structures for electric power distribution in Frankfort a.M., Germany. H. CRAEMER. *Zement* (Germany), May 5, 1932, V. 21, No. 18, p. 263-4.—Brief illustrated description of 2 typical modern r. c. skeleton structures, plant for grinding of powdered coal with r. c. chimney and transformer station.—A. E. B.

Construction on the grid system. S. SZEGO. *Beton u. Eisen* (Germany), April 5, 1932, V. 31, No. 7-8, p. 122-126.—Patented system used for reinforced concrete framing of slabs, arched roofs, domes, etc., consists of slab carried by beams in 2 directions about 5 ft. o. c. Beams intersect carrying walls or girders not at 90° but at 45°. Lower moments and smaller deflections are claimed.—A. A. B.

Buildings for Tubingen University. M. FAISSLER. *Beton u. Eisen* (Germany), May 5, 1932, V. 31, No. 9, p. 134-137.—Description of boiler house and central heating plant construction. Overall size of building is 168 ft. x 65 ft.—A. A. B.

MISCELLANEOUS

Fundamental discussion of German specifications for reinforced concrete of 1932. STEPHAN SZEGO. *Zement* (Germany), May 5, 1932, V. 21, No. 18, p. 259-63.—Author illustrates changes brought about by recent issue of new standard specifications discussing especially simple and cross-wise reinforced slabs, economic design of mushroom ceilings, rib slabs, supports, r. c. columns, electric welding of reinforcement and quality requirements of new regulations—A. E. B.

Bestimungen des Deutschen Auschusses fur Eisenbeton (Designations of the German Committee for Reinforced Concrete) 1932. Wilhelm Ernst u. Sohn, Berlin, 1932, 2 R. M.—New German concrete code appeared in its final form in April, 1932, after tentative draft had been under discussion for a year. It takes account of developments made since previous code (1925) appeared—A. A. B.

Deutscher Auschuss fur Eisenbeton (German Committee for Reinforced Concrete) No. 67. Shear tests of r. c. beams. GRAF. Wm. Ernst u. Sohn, Berlin, 1931. Reviewed in *Beton u. Eisen* (Germany), March 5, 1932, V. 31, No. 5. —Tests were made in Stuttgart in 1929-1930, with special reference to shear requirements of 1925 German Code. Inadequacy of merely placing computed shear reinforcement in members without regard to position is shown.—No. 69. K. HAGER AND E. HENNING. Wm. Ernst u. Sohn, Berlin, 1931, reviewed in *Beton u. Eisen*, June 5, 1932, V. 31, No. 11.—Shear and permeability tests of various construction joints. —A. A. B.

Tables for the calculation of continuous beams. L. KAARMAN. 1931, Part 1, Edition C, 128 p. Reviewed in *Tonind. Ztg.* (Germany), May 30, 1932, V. 56, No. 44, p. 568.—This volume deals with beams with 2 or 3 spans and uniformly distributed load. Clear and simple presentation of material makes book valuable. —A. E. B.

Distributing beams. C. OSTENFELD. Julius Gjellerup, Copenhagen, 1930, reviewed in *Beton u. Eisen* (Germany), May 5, 1932, V. 31, No. 9.—A study of analysis of framing grids in which cross-beams act as load distributors to main girders. Analysis neglects torsional moments and uses as unknown the deflection of joints. A. A. B.

Charts for reinforced concrete slab design. H. LOEW. *Beton u. Eisen* (Germany), May 5, 1932, V. 31, No. 9, p. 141-142.—Two charts for determining dimensions and steel for r. c. slabs. Stresses considered are for steel 16,800 and for concrete 140 to 980 p.s.i.—A. A. B.

End moments in plates supported on four sides. H. CRAEMER. *Beton u. Eisen* (Germany), March 20, 1932, V. 31, No. 6, p. 95.—Study of torsional and shearing forces acting along edges of plates through rational consideration rather than by mathematical analysis.—A. A. B.

Retaining wall formulas. H. NITZSCHE. *Beton u. Eisen* (Germany), June 5, 1932, V. 31, No. 11, p. 174-176.—Development of formulas for design of gravity walls, with and without surcharge, for various factors of safety against overturning and for various minimum compressive stresses at base edge.—A. A. B.

Slabs and structures with thin partitions. WILHELM PETRY. *Preprint, First Congress, Int. Assn. for Br. and Str. Eng.*, Paris, France, May, 1932. 36 p.—Following cases are treated separately: Slabs connected to each other and lying in different planes; circular shells (shell cupolas with circular ground plan) and half cupolas; cylindrical shell roof with cross reinforcement; polygonal cupolas, built up of reinforced cylindrical shells; doubly bent, reinforced shell roofs; principle of compensating static masses. Problems arising in individual types of construction which partly require still further research and experimental work, are briefly mentioned, and examples show development and recent application of these flat supporting surfaces in reinforced concrete buildings.—AUTHOR'S SUMMARY.

Seawall near Breslau. E. WIESNER. *Beton u. Eisen* (Germany), March 20, 1932, V. 31, No. 6, p. 91.—In harbor and dock construction on Oder river at Poepelwitz, 3 types of wall are used: a paved revetment, gravity wall with backstays and combination of paved revetment with rigid frame super-structure carrying gantry cranes and railroad tracks.—A. A. B.

Tallest monolithic reinforced concrete chimney in Europe. K. DEININGER. *Beton u. Eisen* (Germany), April 5, 1932, V. 31, No. 7 and 8, p. 113-116.—Erected by Weiss and Freytag on Heine system, stack at Neuhof plant of Hamburg Elektrizitäts-Werke A. G., has a total height of 492 ft. and clear top diameter of 18 ft. The wind pressure designed for is 45 lbs. per sq. ft. Under this pressure the max. and min. loads on foundation (octagon form 62 ft. diam.) piles is 52 and 5 tons respectively. —A. A. B.

Reinforced concrete rifle range. O. WEDEGAERTNER. *Zement* (Germany), June 2, 1932, V. 21, No. 22, p. 324-5.—Well designed structure is made entirely of r. c. —A. E. B.

Cast iron pipe for columns and column cores. *Eng. News Record*, June 9, 1932, V. 108, No. 23, p. 827.—In tests at Armour Insitute of Technology made to verify claim that cast-iron pipe made by centrifugal process is specially suitable for columns, 10 pipes were tested directly, and 10 as cores of r. c. columns. Length of column and reinforcing were the variables.—G. M.

Design of concrete mixtures. *Nat. Sand Gravel Bul.*, July, 1932, V. 13, No. 7, p. 16-18.—While recognizing importance of flexural strength of concrete, attention is called to fact that, under water-cement-ratio trial method of design recommended by F. H. Jackson (cf. following abstract), 2 concretes having same flexural strength are considered of equal value, even when having widely different compressive strengths, densities, water ratios, cement contents, and probably widely different volume changes, permeabilities, elasticities and other characteristics.—P. McK.

ROADS AND PAVEMENTS

Designing concrete paving mixtures by the water-cement ratio trial method. F. H. JACKSON. *Nat. Sand and Gravel Bul.*, May, 1932, V. 13, No. 5, p.

11-19.—Describes method of application of water-cement-ratio trial method of designing concrete pavement mixtures. Bases determination of proportions on adequate flexural strength with definite limitations on quantity of mixing water which may be used to insure satisfactory durability of concrete. Many factors need be taken into account to insure equitable administration of method. Technique of method discussed in detail from practical as well as economic viewpoint.—P. McK.

ARCHITECTURAL DESIGN

Insulation with light-weight units appeals to Canadian architects. *Concrete*, June, 1932, V. 40, No. 6, p. 12-13.—General use of light-weight concrete units in eastern Canada dates from installation of several plants for production of light-weight aggregates several years ago. One group of university buildings has haydite units for backing up all exterior walls, and in all partitions. A country club has exterior walls 12 in. thick, made up of combination of 8-in. back-up units and 4-in stone facing up to main floor level. Above that exterior walls consist of a combination of 8-in. and 4-in. concrete units, with stucco applied directly. Interior is plastered directly to units. One of Montreal's best appointed apartment buildings has partitions and exterior wall back-up units of haydite concrete, selected mainly for insulating value. Three high-grade buildings occupying adjoining lots in Toronto, all built in 1929, utilize 12-in. and 8-in. cincrete units in backing up exterior walls, and partition units in addition.—C. B.

FIELD CONSTRUCTION

BRIDGES

Arresting abutment shifting on a bascule bridge. O. H. PILKEY. *Eng. News Record*, May 19, 1932, V. 108, No. 20, p. 725-6.—Settling and shifting of abutments riverward on double-leaf bascule crossing of Swan Creek, Toledo, O., necessitated extending bases of abutments and installing 3 concrete struts entirely across river in about 16 ft. of water. The movement consisted of a horizontal drift of about 0.9 ft., a rotation of about 1 deg., and a vertical settlement of over 1½ in. Since the bottom of each abutment was only 12 ft. below normal water level, struts were framed into extensions of bases. Struts are 51 ft. long and 4 ft. deep. Center one is 16 ft. wide, and 2 outer ones are each 9 ft. wide.—G. M.

Repairs to Goeltz Valley viaduct. WEISS. *Beton u. Eisen* (Germany), April 5, 1932, V. 31, No. 7-8, p. 116-119.—A massive stone structure with appearance of a Roman aqueduct, 3-story Goeltz viaduct has received its first major repairs since completion in 1851. Details: length 1880 ft.; width 25 ft.; elevation of roadway above valley 260 ft.; masonry 177,000 cu. yds. Weight of modern trains necessitated reconstruction of roadbed for 2 tracks carried by structure. Old bed, supported on granite slabs was replaced by a new bed on r. c. construction.—A. A. B.

BUILDINGS

Slab strengthening with concrete gun. KICKELHAYN AND AMOS. *Beton u. Eisen* (Germany), April 5, 1932, V. 31, No. 7-8, p. 120-121.—A r. c. slab-and-girder building built in 1911 for 60 lb. per sq. ft. live load, was desired for industrial purposes requiring 150 lb. per sq. ft. live load. After some satisfactory tests the concrete was chipped away till all steel was bare. New reinforcement was then fastened to old and concrete shot against chipped surface. Original slab depth was 4 in.; final depth 6 in. At points of high moment in beams and slabs new soffits were haunched. Repairs involved slab area of 65,000 sq. ft.—A. A. B.

Smooth finish on exposed concrete obtained with plywood. *Concrete*, June, 1932, V. 40, No. 6, p. 17-18—Concrete surfaces were formed with plywood in new r. c. pachyderm building at Chicago Zoological Park. Both exterior and interior surfaces are left as they appeared when forms were removed. Form material consisted of sheets of 5-ply Douglas fir plywood $\frac{9}{16}$ in. thick, used structurally as sheathing material, reinforced by 2 x 4-in. and 2 x 6-in. studding and joists and usual horizontal walers. Assembly of forms was speeded by use of large sheets. Form marks occurred only at vertical lines where sheets were butt-jointed.—C. B.

Standard specifications for building materials and construction. DAVID H. MERRILL AND THEODORE C. COMBS. Published by Sun Printing and Publishing House, San Bernardino, Calif., 550 p.—A compilation made upon popular demand to make readily available in book form complete set of standards on building materials

and construction, for use as reference. Standards included verbatim are those adopted by Pacific Coast Building Officials Conference, and which form a part of the Uniform Building Code.—D. F. J.

Small buildings made of cement blocks. *Zement* (Germany), June 2, 1932, V. 21, No. 22, p. 326.—Section of German Building Exhibition in Berlin features small dwelling houses made of structural parts of concrete and light-concrete (aerokret and gas-concrete).—A. E. B.

Porous light-weight concrete for long-span floors. A. G. HILLBERG. *Eng. News Record*, June 2, 1932, V. 108, No. 22, p. 799-800.—The Grand Gardens Apartments at Fleetwood, N. Y. were constructed with aerocrete floor systems following tests on a model panel. I-beam joists were placed 2 ft. 10 in. to 3 ft. 6 in. apart, and spans varied from 12 ft. 3 in. to 19 ft. 9½ in. Reinforcement was 4 x 12-in. welded wire mesh. Aerocrete was allowed to rise from 3½ in. to depth of 7½ in., giving a weight of about 45 lb. per cu. ft. Field test on completed structure indicated a deflection-span ratio of 1:1900 under a live load of 60 lb. per sq. ft., or 1½ times the design load.—G. M.

DAMS

The Bleiloch dam near Saalburg in Thuringia, Germany. KOEHLER. *Deutsche Wasserwirtschaft* (Germany), 1932, No. 1-3, 17 p.—Detailed description of design and construction of new German gravity dam. Concrete distributing bridge of ingenious design was used. Operations include excavating and concreting jobs, dam and auxiliary structures, water conduits and power station.—A. E. B.

The concreting bridge for the erection of the Bleiloch dam, Germany. *Tonind. Ztg.* (Germany), May 9, 1932, V. 56, No. 38, p. 496-7.—Concreting bridge used for distribution of 2550 cu. ft. of concrete per hr. for construction of dam 213 ft. high and 690 ft. long is erected upon 6 concrete columns 82 ft. apart. Bridge is of 2-story type. Lower roadbed carrries 4 mixers and aggregate bins; 2 tracks for transportation of structural materials and 1 track for cable crane are supported on upper level. Columns remain in main body of dam, separated by expansion joints of special profile which insures firm hold. Cable way distributor is used for erection of power station, about 25,000 cu. yds. of concrete.—A. E. B.

The damages of the Doerverden weir, Germany. ODENKIRSCHEN. *Bautenschutz* (Germany), March 5, 1932, V. 3, No. 3, p. 36-40.—Reconstruction of old weir revealed considerable damage of steel and r. c. structures due to mechanical action of water and sand and chemical disintegration of concrete by action of high content of magnesium sulfate in ground water. Concrete was completely replaced. New concrete was prepared from carefully selected cement and aggregates with admixture of trass.—A. E. B.

MISCELLANEOUS

Production tests of mixers. G. GARBOTZ AND O. GRAF. V. D. I. Verlag Berlin 1931, reviewed in *Beton u. Eisen* (Germany), May 5, 1932, V. 31, No. 9.—The first report of recently founded Research Institute for Construction Machinery. Various types of mixers were tested and reported on from viewpoint of concrete engineer and of mechanical engineer.—A. A. B.

Belt conveyor theory and practice. G. G. DODGE. *Rock Products*, April 9, 1932, V. 35, No. 7, p. 78-79.—In treating power requirements of belt conveyors and classifying items of operation which require power, tables are given for idler spacing and power coefficients, also formulas for determining power required under varying conditions.—E. S.

Watertight forms essential for good concrete work. GEORGE HOCKENSMITH. *Concrete*, June, 1932, V. 40, No. 6, p. 25-26.—Concrete, if placed and tamped properly in watertight forms of correct design, has a smooth surface covered with a thin film of mortar. Breaking this film by rubbing exposes pores and leaves concrete liable to disintegration. Disintegration also follows forms built with cracks in corners and between boards, permitting mortar and water to leak out. Pinholes 4 or 5 in. deep are seen by aid of magnifying glass. Leakage is prevented and dense surface assured by lining forms with ¼-in., 3-ply veneer.—C. B.

Construction control for concrete jobs. A. BREBERA. O. Gracklauer, Leipzig, Germany. Reviewed in *Zement* (Germany), June 23, 1932, V. 21, No. 25, p. 372.—In illustrating importance and necessity of construction control it is pointed

out how quality and strength of concrete and r. c. can be improved. Various materials for concrete production and their properties, testing and specifications are described. Last chapter deals with effect of aggregates upon quality and economy of concrete for road construction.—A. E. B.

Concrete made on construction site must equal ready-mixed. ROBERT C. JOHNSON. *Concrete*, June, 1932, V. 40, No. 6, p. 30.—Contractor, to maintain his position as manufacturer of concrete on job, must be as efficient as ready-mixed concrete man, and must make the same economies. Recent developments are high-early-strength concrete, special-use concrete, and many offshoots of so-called 1:2:4 mixes. One of large developments to come will be use of special cements for special concrete requirements. Contractors must watch for these cements, know how to use them and how to sell them.—C. B.

Ready mixed concrete plant of Consolidated Rock Products Co., Los Angeles, Calif. EDMUND SHAW, *Rock Products*, June 18, 1932, V. 35, No. 12, p. 52-55.—Company has 4 stations at which its truck mixers are charged, one at aggregate plant in city and others at bunkers to which aggregates are shipped from company's plants. Cement is added by sack according to Los Angeles ordinance. Aggregates are charged from weighing batchers and water metered in from calibrated tank. City paving concrete is mixed so grading will plot as definite curve derived from Fuller's curve by applying it experimentally to local materials. Grading is figured from analyses of separate materials in plant bins.—E. S.

Adds second ready-mixed-concrete plant to supply Albany, N. Y. area. *Pit and Quarry*, V. 24, No. 2, Apr. 20, 1932, p. 39-41.—Ready Mix and Supply Co. has adjacent but distinct units with combined capacity of 120 cu. yd. per hr., using a 2-min. mix.—A. J. H.

Baltimore's new 1000-cu. yd. ready mixed concrete plant. *Rock Products*, May 7, 1932, V. 35, No. 9, p. 66-72.—New plant of Arundel-Brooks Concrete Corp., Baltimore, Md., produces 1000 cu. yd. concrete daily. Sand and gravel are received in scows, rewashed, and stored in large concrete bins from which they are sent to the mixing plant by a conveying belt. Cement is dumped in a loading house entirely enclosed. From hopper below track cement is drawn out by screw conveyors. At mixing plant aggregate is kept in 5 compartment bin. Cement feeders are stopped automatically by 6000-lb. dial scale with photo electric control. Sand and gravel are weighed out on a 20,000-lb. beam scale.—E. S.

Some requirements of a purchase specification for ready-mixed concrete. R. B. YOUNG. *Nat. Sand Gravel Bul.*, June, 1932, V. 13, No. 6, p. 3-7.—Ready mixed concrete purchase specification should include consideration of materials, proportions, consistency, measurement, and inspection, and the 2 systems, central-mixing and truck-mixing, should be considered in specifications for mixing and delivery. Tests and acceptance should be covered by definitely stating: where and when concrete is to be sampled; how test specimens are to be made, cured and tested; when tests indicate that specification is met; what procedure shall be followed in case tests indicate specification has not been met; and method of determining volume of batch of wet concrete.—P. McK.

Bulk cement railway hopper cars unloaded by gravity. *Concrete*, July, 1932, V. 40, No. 7, p. 34-35.—Addition of a watertight roof, a transverse bulkhead at center, installation of 4 hopper outlets fitted with slide gates, and accessory fittings, are only changes made by eastern railways to convert hopper coal cars to bulk cement cars, at average cost of $500. After spotting 2 of 4 outlets over screw conveyor, a storm cover is raised and held in position by a steel catch attached to side of hopper. Seal is then broken and sliding gate opened. After half is unloaded, car is pinched ahead to spot second pair of hoppers.—C. B.

Electrical heating of concrete. A. BRUND AND H. BOHLIN. *Beton u. Eisen* (Germany), May 5, 1932, V. 31, No. 9, p. 138.—To heat poured concrete, to achieve early strength in cold weather, electrical method avoids many objections present in other heating arrangements. Electrodes consisting of sheet metal strips are placed in contact with opposite surfaces of concrete member. Resistance of concrete to passage of current (A. C.) generates desired heat. Power consumption in k. w. h. per deg. C. heating of one cu. m. varies from 0.7 to 1.4. Time required to heat concrete to about 40° C. above outside temperature is 4 to 8 hrs.—A. A. B.

The application of spun concrete for public and industrial structures. ORLER. *Il Cemento Armato* (Italy), 1932, No. 2, 4 p.—A description is given of

design, manufacture and application of spun r. c. structural parts such as columns for light-weight ceilings, bridges, electric power lines and piles for foundations.—A. E. B.

Embedded stone concrete produces low-cost walks and floors. JOHN CALHOUN. *Concrete*, July, 1932, V. 40, No. 7, p. 14.—Forms and grade are prepared as usual. Stones 12 to 15 in. in size are laid upon subgrade by hand about 1 in. apart with tops about 1 in. below finished surface. Reinforcement if used is placed upon stones. If topping course is to be used, concrete is run over the stones to the concrete grade and then tamped with a stick slightly less in thickness than the distance between stones, and toppnig placed. Temperature joints may be placed farther apart than with ordinary concrete construction.—C. B.

Damages to structures. CHRISTOFORO RUSSO, F. HAEUSLER AND KARL SCHAEFER. 1932, R. Oldenbourg, Munich, Germany. Reviewed in *Zement* (Germany), June 23, 1932, V. 21, No. 25, p. 372.—Book presents valuable information concerning reconstruction jobs, especially restoration of old damaged foundations of concrete and r. c.—A. E. B.

Fire damage and reconstruction of r. c. docks. H. CANTZ. *Beton u. Eisen* (Germany), March 20, 1932, V. 31, No. 6, p. 88.—Quay warehouse 5 in Stettin, built in 1909 and consisting of r. c. basement and first floor slab with steel and wood roof framing above, was damaged by fire in 1931. Floor slab is formed by r. c. arches of 15 ft. span, supported by beams and tied columns. Transverse cracks were observed in intrados fireproofing and longitudinal cracks in the extrados at crown. Impact of water stream caused spalling of concrete, especially at edges of beams and columns and at expansion joints. Concrete basement slab and walls suffered little damage. Repairs included chipping away all concrete affected and repouring at these points. —A. A. B.

Oberhasli hydroelectric plant. A. LEON. *Beton u. Eisen* (Germany), April 5, 1932, V. 31, No. 7-8, p. 108-112.—First section of Swiss project now nearing completion, is designed for 120,000 H. P. Second section will deliver 160,000 H. P. The two sections have heads of 1790. ft. and 2190 ft. Upper storage area is formed by Lake Grimsel into which projects Lower Aar Glacier. Lake surface is 6280 ft. above sea level. To insure safety against avalanches and temperature change, the drop to turbines takes place in shafts blasted in rock and lined with concrete pipe (6.9 ft. in diam. and 4 to 5½ in. thick). The cost of first section is 82,000,000 Swiss francs.—A. A. B.

Increasing the bearing power of clay soil. D. E. MORAN AND F. O. DUFOUR. *Eng. News Record*, May 19, 1932, V. 108, No. 20, p. 726.—A 9-in. r. c. floor with an area of 55,000 sq. ft. carrying a load of 1¼ tons per sq. ft. was successfully lifted a max. of 8 in. by forcing grout beneath it. A grouting machine of 4-cu. ft. capacity operating under a pressure of 100 lb. was used to force a 1:2 mix beneath floor. To prevent further settlement grout was forced through clay and decomposed rock on which building rested to bedrock. Holes were 7½ ft. o. c., and depth to rock was 18 ft.—G. M.

Cylinder underpinning checks sinking foundation. HARRY SPILLMAN. *Eng. News Record*, April 14, 1932, V. 108, No. 15, p. 544-5.—Settlement to a max. of 15¼ in. of 40,000 kva. power plant of Continental Motors Co. at Muskegon, Mich., was apparently caused by insufficient spacing between piles. Open-ended sections of ⅜-in. steel pipe 16 in. in dia. were driven in open spaces in foundation system, evacuated by compressed air jets, filled with concrete, and then wedged against structure, to reduce rate of settlement rather than to stop settlement entirely at points underpinned.—G. M.

Bottom heading driving on C. P. R. tunnel in Quebec. THOMAS F. RUSSELL *Eng. News Record*, May 19, 1932, V. 108, No. 20, p. 716-8.—Combination of full-width bottom-heading driving, scraper-mucking, and placing concrete lining along with excavation work permitted completion of 5330-ft. tunnel at Quebec by Canadian Pacific Railway in 11 months. Width is 16 ft. and height 22 ft. 6 in. Walls are vertical and roof is a semicircular arch of 8-ft. radius. In self-supporting rock, concrete lining is 9 in. thick on the walls, and 12 in. on roof Under less favorable conditions thickness is increased to 27 in. Concreting operations were started from 3 intermediate positions and from both ends. Bench walls 4 ft. high were poured by hand after which remainder of lining was placed by pneumatic machines.—G. M.

ROADS AND PAVEMENTS

Use and handling of bulk cement on concrete highway work. *Concrete*, June, 1932, V. 40, No. 6, p. 23-25. At least 5 state highway departments use bulk cement in all or nearly all concrete highway construction. One state specifies bulk cement on all concrete paving work, 17 others have used some bulk cement, making a total of 24 out of 43 states reporting, or 56%, with experience in this direction. Nineteen have not used bulk cement, 17 have reported no experience with cement in this form, and 2 actually forbid its use. Details of equipment and methods employed in various states are outlined.—C. B.

Cure concrete highway slabs with heavy kraft paper. MARK MORRIS. *Concrete*, June, 1932, V. 40, No. 6, p. 29.—Two types of duplex papers were tested and found satisfactory in pavement curing by Iowa State Highway Commission. Each was reused at least 5 times without serious depreciation in efficiency. At each application samples were given rigorous laboratory test to determine ability to prevent loss of moisture from mortar cured for first 24 hr. after placing, under wet burlap. Subjected to atmospheric conditions in which temperature of air was 93 to 98° F. and relative humidity 33 to 35, paper showed, initially, very high efficiency in prevention of moisture losses and progressively lower efficiency for each subsequent application. Satisfactory curing requires at least 80% of original water content be retained at the age of 6 days. Papers showed losses of 0.61 to 4.29% at initial application and 2.00 to 7.87% after last application.—C. B.

Some developments in portland cement concrete road construction. RONALD BUTLER HINDER. *Paper 399, Syndey Division, Institution of Engineers, Australia*, March, 1932, 16 p.—Primary object of paper is to explain and investigate merits of system of road construction which will provide relatively cheap roads of high class using Australian-made material. Method considered is known as cement penetration, or grouting system. Complete description of this method, together with numerous test results, costs, and practical construction information, is given. Specifications for the construction of r. c. pavements by 2 grouting systems are given as appendices.—AUTHOR'S SUMMARY.

Survey indicated doweled pavement joints of doubtful value. *Eng. News Record*, March 31, 1932, V. 108, No. 13, p. 476.—The report of the committee on reinforced-concrete pavements and bases acting for American Road Builders Assoc. points out possibility that dowels, as ordinarily used, are not functioning as expected. Dowels are used by 39 states under variety of conditions. Questionable efficiency is obtained by their use as means of maintaining slab alignment and transference of load, due to high intensity of bearing in concrete at joint edge and bending in dowel. Dowel spacing should not exceed 24 in. Some designers meet problem by strengthening slab edges with extra end bars or extra end sheets of reinforcing.—G. M.

Concrete-road resurfacing slab laid on sand cushion. C. S. MULLEN. *Eng. News Record*, March 31, 1932, V. 108, No. 13, p. 461-2. In reconstruction of 4.42 miles of pavement on old Virginia highway between Lynnhaven Inlet and Cape Henry, a 4-in. r. c. slab was laid on 2-in. sand cushion, and pavement widened from 18 to 22 ft. One 200-ft. section was laid directly on old concrete pavement without sand cushion, and 500-ft. section was laid on bituminous mastic mixture of tar with sand, thoroughly compacted by rolling. High-early-strength cement was used for base and headers throughout and for slab at both ends of project. Base course and headers were poured first to serve as forms for slab. Wire-mat reinforcing was placed at depth of 2 in., with 3-in. gap at 105-ft. intervals. Mix was designed on a cement factor of 1.4, and aggregates were proportioned by weight. Base course consisted of a 1:7½ mix. Inspection about 3 months later revealed several cracks in 200-ft. section where no cushion was used.—G. M.

Time studies of concrete paving bring large savings. P. M. TEBBS. *Eng. News Record*, May 26, 1932, V. 108, No. 21, p. 772-4.—Study of paving operations made by Penn. highway dept. in 1929 and 1930 shows increased speed and efficiency are possible with improved methods. Analysis of distribution of time losses and suggestions for lowering them are given. Number of batches per hour was used as basis of comparison. Records show increase in efficiency in 1930 to be 32% over 1929.—G. M.

Two-course concreting assures exact level of steel. *Eng. News Record*, March 31, 1932, V. 108, No. 13, p. 477-8.—A resume of methods of placing fabricated

reinforcement in concrete pavement includes use of chairs, an overhead installing device, sled installing device, and variations of strike-off methods. Advantages and disadvantages of each are discussed. Some form of strike-off method is favored. —G. M.

SEWERS

Concrete in old outfall sewer in good condition after twenty years. M. J. RUARK AND C. E. KEEFER. *Eng. News Record*, May 19, 1932, V. 108, No. 20, p. 721.—Baltimore outfall sewer, constructed in 1908 of a 1:2:4 hand-mixed concrete with gravel aggregate, was recently opened to provide for additional connections. Concrete was in excellent condition, and has suffered no ill effects from hydrogen sulfide. Average ultimate compressive strength of test specimens taken from sewer was 3745 p.s.i.—G. M.

SHOP MANUFACTURE

Grading aggregate for mechanically compacted concrete pipe. GEO. W. GLEESON. *Concrete*, June, 1932, V. 40, No. 6, p. 7-11.—The following facts on intermittent grading are recommended where resistance to percolation with maintenance of satisfactory quality is desired in manufacture of small pipe: Fine aggregate—durable sand is desired, containing high percentage of material lying on No. 48 and No. 100 sieves with as low a percentage as possible on No. 14 and No. 28 sieves. Coarse aggregate—durable particles passing limits of maximum size desired and containing little or no material on the No. 14 and No. 28 sieves is suitable. A combination of above should be effected so that practically 50% by weight of combination lies above a No. 8 screen, it being feasible to increase maximum size of coarse particles to increase strength as recommended fineness modulus or experience dictates.—C. B.

California products manufacturer uses pumice as aggregate. W. A. SCOTT. *Rock Products*, April 23, 1932, V. 35, No. 8, p. 68-69.—Pumtile is trade name of unit made by Jourdan Concrete Pipe Co., Fresno, Calif. Mix consists of 1 sack cement, 6 cu. ft. pebble pumice, and 20 lb. pumicite with water for a wet mix. Weight is 80-90 lbs. per cu. ft. Pumice and pumicite are from deposits 18 miles north-east of Fresno. Several handsome residences have been constructed of these units. Compressive strength is 1350 lbs. in 28 days and units are stated to have withstood fire tests up to 2000° F.—E. S.

Develop burial vault business in community of ordinary size. C. C. MILLHOUSE. *Concrete*, June ,1932, V. 40, No. 6, p. 21-22. Concrete burial vaults and sepetic tanks make good combination for products manufacturer and concrete contractor. Both employ similar manufacturing processes and vaults are in steady demand while tanks can be developed into profitable line with fluctuating demand. Plant at Chambersburg, Pa., supplies 300 to 400 vaults per year. A mixture of one part of portland cement to three parts of well graded washed sand, running in sizes up to small gravel is used in vault manufacture. Walls are reinforced vertically and horizontally, and bottom lengthwise and crosswise. Vaults are coated with waterproofing, after which surface is finished with thin slush of neat cement rubbed by hand with a gunny sack.—C. B.

Aggregate grading for tamped concrete pipe. GEORGE W. GLEESON. *Rock Products*, April 9, 1932, V. 35, No. 7, p. 70-71.—While pipe has been improved by including larger size aggregate, this practice may have been carried too far. Segregation, spotty pipe, failure to pass percolation tests, and other evils have sometimes resulted. For a correct grading, ratio of surface area to volume is index of proper aggregate size. Surface area-volume ratios have been computed for commercial pipe sizes and plotted against pipe diameters. In small pipe sizes ratio of surface area to volume is well over 2, but decreases until with 30-in. pipe surface area is abour half concrete volume. Resistance to placing concrete is proportional to this ratio, so far as water tightness is concerned. Most trouble from percolation is found in small sizes. Definite relation of fineness modulus to surface-volume ratio has been found. Since such a relation is more useful for pipe diameters, chart gives fineness modulus against pipe diameter.—E. S.

Abstracts

Materials

Admixtures

Use of diatomaceous earth on construction. Victor Charrin. *Genie Civil,* July 30, 1932, No. 2607, p. 111-113.—Masonry of insulating brick of diatomaceous earth requires mortar for joints, in which sand is replaced by ground diatomaceous earth. Mix of 80% diatomaceous earth and 20% portland cement is recommended; least possible quantity of mixing water should be used. This type of brick was used in construction of a soundproof talking picture studio. Admixture of diatomaceous earth to any mortar or concrete helps fill the voids; contains free silica able to combine directly with lime during setting. Such admixtures were studied by the Federation industrielle des Materiaux de Construction on 5 cm. cubes; a gain in strength was observed in every case. A max. admixture of 5% is recommended in France. —M. A. C.

Aggregates

International standardization of sieves. M. Feret. *Rev. Materiaux Construction Trav. Publics,* July, 1932, No. 274, p. 265-268.—Report presented May 14, 1932, to French Association for Testing Materials, discusses gradation of size of opening, diameter of wire, allowable error, perforated sheets and their comparison with wire mesh.—M. A. C.

What are the advantages of the fineness modulus? Ottokar Stern. *Zeitschr. Oster. Ing. und Archit. Vereins* (Austria), May 6, 1932, V. 84, No. 17-18, p. 86-9. —A critical mathematical study introduces new aid for proportioning concrete in theory and practice. Fineness moduli of dry concrete materials enable valuation of maximum degree of density and consistency obtained with any amount of mixing water for which they give a uniform measure.—A. E. B.

The uniform valuation of the fineness modulus for the construction control. Leo Bloch. *Zement* (Germany), June 30, 1932, V. 21, No. 26, p. 383-4.—Author points out difficulties in using fineness modulus for construction control due to different methods of determination in various European countries and proposes adoption of international values obtained on same basis by uniform valuation of sieve curves. A chart and practical method for construction control are presented. —A. E. B.

Experiences obtained from studies with the concrete pump. Kurt Mauthner. *Zeitschr. Oster. Ing. und Archit. Vereins* (Austria), March 11, 1932, V. 84, No. 9-10, p. 46-8.—Purpose of investigation was to study properties of cement and aggregates necessary and favorable to secure trouble-free operation of pump and methods to be applied to improve inadequate aggregates. None of concrete materials should be water repellent. Blast furnace slag cements are not advisable Aggregates with more than 45% or less than 10% of ¼-in. fraction should not be used. Finest particles below 0.008 in. are most important constituents; 8-10% are necessary.—A. E. B.

Some factors affecting comparative strength tests of concrete made with different aggregates. Stanton Walker. *Nat. Sand Gravel Bul.,* July, 1932, V. 13, No. 7, p. 13-14.—Certain factors may affect results of tests made to evaluate one aggregate in terms of another. Studies made thus far have not been conclusive in showing definite relationship between age at test and ratio of concrete strength of one aggregate to another, and quality of cement with which aggregates are tested may have important effect upon concrete strength. Some data along these lines are cited. Preliminary tests, in laboratory of National Sand and Gravel Association on 2 aggregates and 3 cements, show one aggregate to produce 10 to 22% higher flexural strength than other in same concrete mixture, depending upon cement used.—P. McK.

Cement

Fine grained cement. D. Steiner. *Tonind. Ztg.* (Germany), June 20, 1932, V. 56, No. 50, p. 638.—Brief discussion of relationship pointed out by Eiger—cf.

Tonind. Ztg. (Germany), May 23, 30, 1932, V. 56, No. 42, 44, p. 532-3, 558-60— between cement strength and most favorable amount of mixing water for hydration of cement. Curves presented by Eiger are critically examined.—A. E. B.

Air separator device determines fineness distribution. ORLA A. LARSEN. *Concrete*, July, 1932, V. 40, No. 7, p. 41-42.—Apparatus consists of large glass tube, connected to a small blower, which blows air current through receiver up the glass tube. Cement is charged into tube, where coarse particles are held in suspension by air current while particles finer than particle size at which separation takes place are carried out of glass tube.—C. B.

The Kelly tube and the sedimentation of portland cement. CHARLES G. DUNCOMBE AND JAMES R. WITHROW. *Journal Phys. Chem.*, Jan. 1932, V. 36, No. 1, p. 31-51.—Sedimentation apparatus designed by Kelly to determine fineness graduation of cement and particle sizes has a number of disadvantages which are overcome by designing a modified Wiegner tube which gives very accurate results. It is easy to handle, eliminates evaporation loss, has almost no lag and allows use of viscous liquids which are especially desirable for coarse cements.—A. E. B.

Measurement of particle size with an accurate air analyzer; the fineness and particle size distribution of portland cement. P. S. ROLLER. *Am. Soc. Testing Materials Preprint*, 1932, 19 p.—Essential new feature of apparatus for determining particle-size distribution of fine powder by air separation and weighing of fractions is that a U-container undergoes constrained oscillations so charge of powder is continuously brought into effective contact with air jet by undergoing a combined translatory and rotatory motion. Analysis into fractions 0 to 5, 5 to 10, 10 to 20, 20 to 40, 40 to 80, and 80 to (160) microns has been made of 4 brands of portland cement and of 3 laboratory ground clinkers. Deviation of recorded percentages is nearly same for all fractions and averages close to $\pm 0.2\%$. A rapid and efficacious method is described of preparing fine powder for microscopic examination by using a thin platinum wire and a drop of 0.25% solution of saponin in 50% alcohol. Distribution of particle sizes in each fraction is closely within theoretical limits given by Stokes' law and mean diameter may be taken as theoretical mean of Stokes' law limits. Surface mean diameter and dispersion of cements are calculated, and the results for percentage by weight in each fraction are plotted so that area under curve represents surface area of cement powder contained between any 2 particle sizes. Grinding selectively segregates chemical constituents of a portland cement into 2 major particle-size categories.—AUTHOR'S SYNOPSIS

"Albesco," a white portland cement. RICHARD HOFFMANN. *Zement* (Germany), June 30, 1932, V. 21, No. 26, p. 382.—A new white portland cement produced at plant in Koenigshofen-Tschischkowitz, Czecho Slovakia, was tested at 4 institutes for testing materials for weight per cu. ft., fineness, setting time, soundness and tensile and compressive strength. Latter correspond with those of good portland cement.—A. E. B.

Slag cement. The cement for massive work. G. DUPONT. *Ciment*, June, 1932, V. 37, No. 6, p. 197-203.—Review of booklet by Cléret de Langavant discusses normal strength, hydraulicity, behavior in presence of destructive solutions, exposure to air, relative suitability for use in grouting, and disadvantages, when used in thin sections; actual strength values and proportions of slag cement concrete used on some of the large jobs in France, such as Eguzon and Guéledan dams, public works of City of Paris, including subway located in part below hydrostatic level of river Seine. Erroneous idea that slag cement is destructive to steel reinforcement is refuted emphatically on basis of observations on actual structures over 30-yr. period by French cement company Lafarge et du Teil. In comparison with American sand-cement, economic advantages are stressed.—M. A. C.

Yugoslav standard specifications for portland cement. *Zement* (Germany), May 12, 1932, V. 21, No. 19, p. 275-6.—New issue of March 1, 1932 specifications include new definition, hydraulic modulus, chemical composition, fineness, setting properties, constancy of volume, tensile and compressive strength requirements, preparation of test specimens, storage and breaking.—A. E. B.

A modern rotary kiln installation. The Lepol Kiln. W. EITEL, ALFRED MUELLER AND K. A. GOSLICH. *Tonind. Ztg.* (Germany), July 4, 1932, V. 56, No. 54, p. 679-84.—A comprehensive investigation of new type of rotary kiln equipped with travelling grate demonstrates superiority to old rotary kiln. Raw materials are granulated with 12 to 14% water and heated upon grate by hot kiln gases. Pre-

heated material is burned in 80 to 100-ft. rotary kiln to clinker of high quality and cooled in special cooler. Installation built in San Sebastian, Spain, is critically examined and complete data on kiln operation, fuel and power consumption, chemical composition of raw materials, coal and clinker, physical properties of cement and heat economy are presented. Heat balance of entire system is worked out.—A. E. B.

New aluminous cement plant in Los Angeles. *Rock Products*, July 2, 1932, V. 35, No. 13, p. 59.—American Aluminous Cement Co. of America, is building plant for making fused aluminous cement in San Pedro, port of Los Angeles, Calif. One water-jacketed shaft furnace has been erected, with lay-out plans for 6. Raw material is bauxite from Italy. Later it is hoped to develop deposits of bauxite on Pacific Coast. Company is making cement under license from holders of French patents.—E. S.

A damp cupboard for cement testing. *Cement and Cement Manuf.* (Eng.), June, 1932, V. 5, No. 6, p. 211-3.—In damp cabinet used at Imperial Institute, official testing station for certification of British cement for export, features are glazed sliding doors to prevent drafts when opened, top lighting, and humidifying device, which consists of jute strips hung loosely over back and side walls and dipping into shallow and narrow water tanks fixed around top of chambers. Lower ends of jute strips dip into g. i. trays in bottom. Upper tanks are kept at constant level and supplied from 50-gal. tank kept at normal testing room temperature.—J. C. P.

The future of the rotary kiln. *Cement and Cement Manuf.* (Eng.), June, 1932, V. 5, No. 6, p. 191-3.—Brief review of development in kiln design, calls attention to new aspect of subject in Geoffrey Martin's book, "Chemical Engineering and Thermodynamics applied to the Cement Rotary Kiln" (Crosby Lockwood & Son). Efficiency is largely dependent on flame temperature, and the higher this is, the lower will be fuel consumption. Losses by internal radiation become serious under these conditions, and economy attained with Lepol type of kiln suggests that further study of 2-stage kiln should be made.—J. C. P.

Heat economy in the cement industry—IV. Hans Bussmeyer. *Cement and Cement Manuf.*, (Eng.), June, 1932, V. 5, No. 6, p. 219-25.—So many variables and conditions influence power and fuel consumption in cement plants that general or average figures cannot be given, in lieu of which detailed data are given in tabular form for 7 typical cement plants, showing power required for different departments, and total fuel consumption per 1000 kg. of clinker produced.—J. C. P.

Grinding plant research—Part VI. William Gilbert. *Rock Products*, July 2, 1932, V. 35, No. 13, p. 35-37.—Results from tests at different mills in commercial operation were not really comparable. It was thought by opponents of plan to use a 48 x 48-in. laboratory mill that closed mill grinding definite weight could not approximate work of mill with continuous feed. Tests were made on a 15-ft. tube mill with taking continuous feed from kominutor at rate of 3360 lb. per hr. Test made on clinker showed residue of 19.2% on 180 mesh with continuous feed and 20% with closed mill, time being calculated from hourly capacity.—E. S.

Hydrothermal synthesis of calcium hydro-aluminates. Shoichiro Nagai. *Cement and Cement Manuf.* (Eng.), June, 1932, V. 5, No. 6, p. 205-10.—Mixtures of lime and aluminum hydroxide, and of lime and alumina in various molecular proportions were heated in saturated steam in an autoclave at pressures of 5, 10 and 20 kg. per sq. cm. for 24 hr. Analyses of resulting products showed development of following hydrates, $C_3A \cdot 6H_2O$, $C_2A \cdot 7H_2O$ or $C2A \cdot 6H_2O$, and $C_2A \cdot 3H_2O$. First coincides with that obtained by Thorvaldson in the hydration of C_3A, last is obtained from hepta- or hexa-hydrate of di-aluminate by heating at higher temperature and pressure.—J. C. P.

Hydration of aluminous cement. K. Koyanagi. *Concrete*, Aug. 1932, V. 40, No. 8, p. 40-46.—In hydration of aluminous cement "ciment fondu," a small quantity of lime hydroxide is first formed by decomposition of calcium silicate contained in cement, and this lime combines with monocalcium aluminate to form hydrate $2CaO \cdot Al_2O_3 \cdot 7.5H_2O$. Reaction is very vigorous if enough lime is in solution; in small quantity and comparatively slowly in hydration of ciment fondu. Two other reactions follow, monocalcium aluminate giving up a molecule of aluminum hydroxide and changing into hydrate of dicalcium aluminate of same chemical composition as above, and dicalcium hydroaluminate changing into hydrate of tricalcium aluminate.—C. B.

Hardening of portland cement. K. KOYANAGI. *Cement and Cement Manuf.* (Eng.), June, 1932, V. 5, No. 6, p. 214-8.—Tippmann declared that CaO in pure water slakes to an amorphous hydrate with no formation of crystalline hydrate. $CaSO_4$ acts as inciter of crystallization, and hence gypsum solution prevents formation of amorphous hydrate. In very dilute gypsum solution $Ca(OH)_2$ is produced in very fine needles. Author questioned these statements, and his experiments showed that a very fine, slowly developing hexagonal form of $Ca(OH)_2$ is obtained from lime slaked in pure water; that supersaturation quickly ensues on slaking lime in gypsum solutions, which causes more lime to dissolve at first than would dissolve in pure water, but that later solubility of lime is less in gypsum solutions than in pure water; and finally that fine needles described by Tippmann are not crystalline $Ca(OH)_2$, but calcium sulfoaluminate.—J. C. P.

The question about the existence of the tricalcium silicate. ERNST JAENECKE AND R. BRILL. *Zement* (Germany), June 30, 1932, V. 21, No. 26, p. 380-1. —Authors report on X-ray investigations of dicalcium silicate and tricalcium silicate prepared by Bogue and explain why they had obtained different patterns with their own synthetically prepared materials. They fully agree with other investigators about existence of tricalcium silicate.—A. E. B.

The system CaO-$2CaO.SiO_2$-CaF_2 and remarks about the "Alit." ERNST JAENECKE. *Zement* (Germany), June 30, 1932, V. 21, No. 26, p. 377-9.—Various cooling curves of system CaO—$2CaO . SiO_2$—CaF_2 were experimentally investigated to contribute to question of existence of tricalcium silicate. Experiments however could not be applied to this question. A new explanation for formation of solid solutions in ternary system CaO—Al_2O_3—SiO_2 is given and relationship noted between these solid solutions and "Alit."—A. E. B.

Studies on the hydrothermal synthesis of calcium aluminates and silicates between lime and alumina or kaolin—Part 2. SHOICHIRO NAGAI. *Kogyo Kwagaku Zasshi, Journal of Soc. Chem. Ind.* (Japan), June 1932, V. 35, Supplemental binding No. 6, p. 256-60B.—Continuing previous studies—cf. *Kogyo Kwagaku Zasshi, Journal of Soc. Chem. Ind.* (Japan), 1932, V. 35, p. 537—author treated a mixture of lime and kaolin of a composition approximately that of ordinary portland cement clinker by (1) hydrothermically heating under ordinary pressure and (2) ordinary dry heating for 1 hr. at temperatures 1292 to 2012° F. Degree of combination was observed by determination of uncombined lime. Uncalcined kaolin combined with CaO by hydrothermal heating under pressure, calcined kaolin combined in larger amounts.—A. E. B.

Studies on hydrothermal synthesis of calcium silicates under ordinary pressure—Part 4. SHOICHIRO NAGAI. *Kogyo Kwagaku Zasshi, Journal of Soc. Chem. Ind.* (Japan), July 1932, V. 35, Supplemental binding No. 7, p. 320-6B.—Continuation of previous studies—cf. *Kogyo Kwagaku Zasshi, Journal of Soc. Chem Ind.* (Japan), Oct., Nov., 1931, V. 34, Supplemental binding No. 10-11, p. 378-81B, 418-22B; Apr. 1932, V. 35, Supplemental binding No. 4, p. 153-6B; JOURNAL A. C. I., Feb., Sept., 1932, V. 3, 4, No. 6, 1, *Abstracts*, p. 129,—gives detailed results of (1) prolonged heating of mixture of $1CaO : 1SiO_2$, (2) heated products from mixtures $1.25CaO : 1SiO_2$ and $1.75CaO : 1SiO_2$, (3) fractional dissolution of various silicates and (4) determination of percentages of silicates in these products of fractional dissolution method.—Part 5. *Ibid*, Aug. 1932, V. 35, Supplemental binding No. 8, p. 380-4.—Results of studies on (1) silicates produced by heating hydrothermically mixture $3CaO : 2SiO_2$ from 1652 to 2192° F., (2) comparison of silicates produced by heating latter mixture hydrothermically at 2192° F. with those produced by dry heating at 2588 and 2642° F. and (3) fractional separation of mixed silicates and determination of their percentage.—A. E. B.

Summary of physical properties of components of cement. D. L. SNADER. *Concrete*, July, 1932, V. 40, No. 7, p. 43-44.—Tricalcium silicate appears to be most valuable component of portland cement. It sets and hardens rapidly, does not undergo volumetric change during storage in damp closet, remains sound, and has high enduring qualities under severe conditions of exposure. Compressive strength is relatively high and increases with age. Dicalcium silicate appears to be a valuable compound even though it is slow setting, has low early compressive strength and slow increase with age. It does not undergo volumetric change in damp closet, remains sound, and has high enduring qualities under severe conditions of exposure. Tri-

calcium aluminate has valuable property of setting and hardening rapidly and has relatively high compressive strength; but under action of contact water undergoes large increase in volume and disintegrates rapidly and completely. It may not exist as component of portland cement, or if it does occur as one of components, it must be present only in small percentages. Dicalcium ferrite is volumetrically most unstable of 4 compounds studied. Increase in volume and disintegration are greatly accelerated by contact water. Studies by Colony show it is not a normal component of portland cement.—C. B.

Chemical composition of the liquid phase of the standard paste of portland cement. H. HEIN. *Tonind. Ztg.* (Germany), June 20, 1932, V. 56, No. 50, p. 632-4.—Nature of substances dissolved in gauging water depends chiefly upon ratio of cement and water. Dissolved potassium aluminate and potassium silicate are transformed into potassium hydroxide solutions saturated with calcium hydroxide when no gypsum is present. Alkalies seem to have a speed regulating effect upon chemical reactions during setting process of cement.—A. E. B.

Contributions to the knowledge of high alumina cement. GUSTAV HAEGERMANN. *Tonind. Ztg.* (Germany), June 20, 1932, V. 56, No. 50, p. 635.—Author answers objections by Vierheller—cf. *Tonind. Ztg.* (Germany), May 2, 1932, V. 56, No. 36, p. 470-1. Same retardations of hardening due to high temperatures were observed in standard compression cubes and large cubes of 8 in. size. This phenomina occurs when concrete has reached this temperature before hardening has taken place.—A. E. B.

MISCELLANEOUS

Full-load calibration of a 600,000-lb. testing machine. H. F. MOORE, J. C. OTHUS AND G. N. KROUSE. *Am. Soc. Testing Materials Preprint*, 1932, 5 p.—Calibration of screw-power, balance-beam testing machine of 600,000-lb. capacity was made by use of two 10,000-lb. standard weights and an elastic bar fitted with a delicate strainometer. The elastic bar was used merely as a transfer instrument. All that was necessary was to have a strainometer sufficiently sensitive that load on bar could be reproduced a few moments later with a high degree of accuracy. After 27 years of rather severe service testing machine was found to still have a high degree of accuracy and sensitivity.—AUTHOR'S SYNOPSIS

An automatic autographic extensometer for use in tension tests of materials. R. L. TEMPLIN. *Am. Soc. Testing Materials Preprint*, 1932, 10 p.—Automatic autographic extensometer developed in response to a demand for an inexpensive, accurate method for determining yield strengths of materials in routine tension tests provides a means of obtaining load-strain curves at strain magnifications varying from about 400 to about 16,000. For commercial routine tests magnifications of 400 or 800 are used, and yield strengths can be obtained with an accuracy comparable to the usual values for ultimate tension strength. While instrument has been designed primarily for tension tests, it can also be used for compression tests without modification.—AUTHOR'S SYNOPSIS

PROPERTIES OF CONCRETE

Determination of setting time of portland cement. A. C. DAVIS. *Cement and Cement Manuf.* (England) May, 1932, V. 5, No. 5, p. 166-72.—Methods of British Std. Spec. for setting time and normal consistency, involving use of Vicat needle, are described and discussed, with mention of practical importance of the property and things which affect it. Distinction between time of set and rate of hardening, and need for test to determine more accurately the latter property are emphasized.—J. C. P.

The cement macadam road, investigation of road samples. KNIPPING AND GOELTZ. *Strassenbau* (Germany), 1932, No. 10-11, 7 p.—In connection with construction, designs and principal application for cement macadam pavements, authors report results of investigations undertaken at Technical University in Darmstadt, Germany, to study weight per cu. ft., water absorption, frost resistance, compressive strength and resistance against abrasion of concrete samples taken from highways. —A. E. B.

Physical changes of cement mortar due to the action of carbon dioxide. C. R. PLATZMANN. *Zement* (Germany), June 30, 1932, V. 21, No. 26, p. 381-2.—Transformation of calcium hydrate, liberated in set cement, into crystalline calcium carbonate by action of CO_2 should result in increase in volume of specimens. Actually

shrinkage of test pieces which is due to action of CO_2 upon gel structure of cement and a gel richer in SiO_2 and Al_2O_3 and poorer in CaO remains after chemical actions have taken place.—A. E. B.

Stiffness test of concrete. L. KRUGER. *Beton u. Eisen* (Germany), July 5, 1932, V. 31, No. 13, p. 198-203.—New German Code specifies spread test for job control. Tentative code called for 12-in. cone with upper and lower diameter of 4 in. and 8 in. respectively. It is placed on metal plate and filled in 3 layers, each layer being lightly tamped 10 times with 1 x 1-in. wooden rammer. After removal of cone metal plate (28 x 28 in.) is raised 1½-in. and dropped 4 times and greatest and least diameter of concrete pile measured. Their mean is stiffness factor. Final form of code adopts 8-in. height of cone. The tests described include some with American drop table. —A. A. B.

The relation between compressive strengths of concrete at the ages of 7 and 28 days as measured with test cubes. OTTO GRAF. *Zement* (Germany), June 30, 1932, V. 21, No. 26, p. 386.—Results of 250 tests with 8 and 12-in. cubes indicate that compressive strength at 28 days is at least 1.2 times that at 7 days and not higher than 1.7 times 7 day strength plus 850 lb.—A. E. B.

Modern knowledge about water permeability of mortar and concrete. K. WALR. 1931, Wilhelm Ernst and Sohn, Berlin, Germany, 92 p. Reviewed in *Zeitschr. Oster. Ing. und Archit. Vereins* (Austria), Apr. 22, 1932, V. 84, No. 15-16, p. 84.—Book deals with effect of cement, aggregates, grading, water contents and method of working upon density and water permeability of concrete and gives directions for systematic control of structure of concrete.—A. E. B.

Heat conductance in concrete. *Gewapend Beton* (Holland), 1932, No. 9, 3 p. —In results of experiments to study various heat conductivities of normal and lightweight concrete, relations between heat conductivity number and weight of concrete, are pointed out.—A. E. B.

Chemistry of concrete road construction—Part 1. C. R. PLATZMANN. *Betonstrasse* (Germany), May-June 1932, V. 7, No. 5-6, p. 55-7.—Article deals with general questions of improvement of concrete strength for road work, increase of resistance against abrasion, use of calcium chloride admixtures and chemical hardening of concrete.—A. E. B.

Disintegration phenomina on coke cooling towers of concrete and their prevention. THEODOR KLUG. *Bautenschutz* (Germany), July 5, 1932, V. 3, No. 7, p. 82-7.—Greatest damage is caused by chemical action of sulfur compounds in vapors which cause formation of gypsum and calcium sulfoaluminates. This and mechanical action of great and rapid changes of temperature lead to disintegration. Damage can be prevented by suitable design, use of proper heat resisting aggregates (basalt, blast furnace slag, porphyrus) and protective coatings.—A. E. B.

Experiments to determine the shear strength and water impermeability of concrete in working joints treated in various manners. K. HAGER AND E. NENNING. 1931, Wilhelm Ernst and Son, Berlin, Germany. Publication No. 69 of German Committee on Reinforced Concrete. Reviewed in *Zement* (Germany), June 23, 1932, V. 21, No. 25, p. 372.—Concrete specimens in form of half-cubes made of different kinds of cement were jointed together in various manners. Roughening and wetting of old concrete surfaces on which new concrete is to be placed was unfavorable. Shear strength of 2 fresh concrete specimens was variable and depended upon brand of cement and kind of concrete. Water permeability was studied with specimens of different ages cemented together. Joints were not more impervious than concrete itself.—A. E. B.

Concrete test cubes. W. L. SCOTT. *Conc. Constr. Eng.* (*England*), July, 1932, V. 27, No. 7, p. 374-79.—Criticism of test samples of concrete as a representation of concrete in actual work cites wide divergences between max. and min. results obtained in test cubes. Wood molds should be lined with zinc or other metal. Steel molds should be used wherever possible. If concrete is dry mix, loss of water by saturation by untreated wooden molds is destructive. Test cubes should not be subject to vibration, and should be stored where min. temperature is 40°F.—J. M.

Comparative pull-out tests. G. P. MANNING. *Conc. Constr. Eng.* (England), July, 1932, V. 27, No. 7, p. 380-83.—Bond developed in usual pull-out specimens is higher than in test-beams. In standard pull-out test concrete is in compression and steel in tension while in beam both are in tension. Specimen of pull-out type were devised so both concrete and steel were in tension, to discover reason for discrepancy.

Three specimens of each of 2 types were made and tested—one with concrete in compression and other in tension. Both were 8-in. cubes with ¾-in. steel bars. Tension in surrounding concrete does not decrease grip, hence bond failures of test beams are due to other cause.—J. M.

Effect of vegetable oils on concrete. F. W. FREISE. *Conc. Constr. Eng.* (England), June, 1932, V. 27, No. 6, p. 346-51.—Inspection and laboratory tests using different kinds of oils of Brazilian vegetable oil industry showed that vegetable oils, during manufacturing stages or storage, injured concrete. Fatty acids in vegetable oils reacted on free lime, tending to disintegrate cement and oxidation of fatty acids accelerated disintegration. Close relationship exists between destructive properties of fatty acids and their molecular weights. Some seeds due to ulmic acids developed in moist surroundings are destructive to best concrete surface. Two methods of avoiding or reducing deleterious influences on concrete are choice of special brand of cement or protection of surface. Concrete should be as dense as possible, kept wet as long as possible and allowed to harden properly. Reinforcing elements should have at least 1-in. coverage. Hot solution of silicate of soda applied under pressure is effective against weak solutions of deleterious liquids or vapors. A coating of zirconium oxide is highly effective but too costly to be practical. Floors and passages of concrete can be protected by coatings of asphalt applied hot.—J. M.

The electric strain meter and its use in measuring internal strains. R. E. DAVIS AND R. W. CARLSON. *Am. Soc. Testing Materials Preprint*, 1932, 9 p.—Remote reading electric strain meter, has been developed primarily to determine deformations and temperatures within mass concrete. Its operation for determination of deformation depends upon known fact that within elastic limit electrical resistance of steel wire varies in direct proportion to stress in wire. For determination of temperature well-known relation between resistance and temperature of steel wire is employed. Design is such that temperature observations are not affected by deformation and that strain observations are not affected by temperature changes. Representative test results from strain meters embedded in concrete shown, indicate a close and consistent agreement between strain-meter readings and corresponding readings taken with a mechanical strain gage. Calibration remains constant over considerable periods of time.—AUTHORS' SYNOPSIS

Tests on consistency and strength of concrete having constant water content. INGE LYSE. *Am. Soc. Testing Materials Preprint*, 1932, 8 p.—Test data presented show consistency of concrete remains nearly constant regardless of richness of mix, if type and gradation of aggregates and water content per unit of fresh concrete remain constant. Series of tests including 5 different brands of cement showed brand of cement had a slight effect upon consistency of concrete, while richness of mix for each cement had practically no effect. Series including 9 different concrete aggregates showed type and gradation of aggregates had a great effect upon water requirement for given consistency of concrete, while for given type and grading of aggregates and constant water content consistency was nearly same for all mixes. Since cement and aggregate contents were only variables in these concrete mixes, strength of concrete was necessarily determined primarily by cement content. Relation between cement-water ratio and strength of concrete was a straight line, and therefore relation between strength and cement content was also straight line for mixes of constant water content. Conclusions are: Cement is strength-giving element in concrete. Above a given min. number of cement particles necessary to give workability and binding strength to concrete, strength increases in direct proportion to increase in number of cement particles in a unit of water. For ordinary aggregates, a change in richness of a concrete mix of a given consistency is accomplished by keeping water content constant and substituting 0.85 lb. of aggregate for each pound decrease in the cement content, or *vice versa.*—AUTHOR'S SYNOPSIS

Volume changes of an early-strength concrete. E. R. DAWLEY. *Am. Soc. Testing Materials Preprint*, 1932, 17 p.—This investigation consisted of a study of changes in length accompanying changes in temperature and moisture content of concrete specimens of various mixes and consistencies when made with ordinary portland cement and with a certain early-strength portland cement. Compressive strengths, coefficients of thermal expansion and of moisture expansion, and absorption were determined for various kinds of concrete studied. Early-strength cement produced stronger concrete than ordinary portland cement in equivalent mixes at ages of 3 and 28 days. The w/c ratio-strength relation for early-strength cement is

of a different character from that of ordinary portland cement. Coefficient of thermal expansion was more affected by kind of coarse aggregate used than by kind or amount of cement. Coefficient of moisture expansion increased with increased cement content. For concretes of equal 3-day compressive strength coefficients for early-strength concrete were smaller. For equal 28-day strengths coefficients of early-strength specimens were higher where limestone coarse aggregate was used but kind of cement made no difference in the coefficients when sandstone aggregate was used. Specimens made with early-strength cement absorbed less water than those made with ordinary portland cement. Specimens made with early-strength cement were more durable in freezing and thawing than with ordinary portland cement.—AUTHOR'S SYNOPSIS

ENGINEERING DESIGN

BRIDGES

River Ver bridge, Hertfordshire. W. W. DAVIES. *Conc. Constr. Eng.* (England), July, 1932, V. 27, No. 7, p. 398-405.—Bridge is 70 ft. wide between parapets, span 40 ft., clearance 25 ft. Most important factors in choice of design were: value of span-height ratio, necessity of maintaining full span for height of 10 ft. above ground level, difficulty of raising embankment simultaneously behind both abutments and need for speed in erection. "Flat arch" design was adopted with mass concrete abutments and r. c. beams and deck slab.—J. M.

The Drusus bridge in Bolzano, Italy. MIOZZI. *Annali dei Lavori Pubblici* (Italy), 1931, No. 12, 23 p.—Highway bridge 46 ft. wide erected entirely of r. c. has 3 openings of 72 and 115-ft. spans. Roadbed is supported by transverse beams. Special construction methods were used for center section to reduce stresses.—A.E.B.

The concrete bridges across the Twenthe canal. VAN BERGEN AND MULDER. *De Ingenieur* (Holland), 1932, No. 23, 7 p.—Road and railroad concrete arch bridges described have 3 spans. Superstructures at center spans consist each of 2 main arches with suspended roadway. Construction progress is recorded.—A. E. B.

Large reinforced concrete bridges. H. LOSSIER. *Ciment*, June, 1932, V. 37, No. 6, p. 204-213.—Concrete, reinforcement, phenomena accompanying settling or "sliding" in a concrete arch, erection methods, including Freyssinet arch at Plougastel, reduction of certain parasitic stresses in hyperstatic arches, types of arches, beam bridges, architectural considerations and discussion of life and future make up report presented in *Revue Universelle des Mines*, Feb. 15, 1932.—M. A. C.

Unusual girder contour marks new concrete bridge. H. D. HILBORN. *Eng. News-Rec.*, July 14, 1932, V. 109, No. 2, p. 36-38.—New McKee St. bridge over Buffalo R. at Houston, Tex., represents an unusual solution of problem complicated by several restrictions. First cost, maintenance and appearance each exerted influence in choice of structure. A vertical clearance of 42 ft. specified by War Dept. precluded deck-girder construction and continuous through-girder construction was adopted. Depth of girders at any given point is proportional to ordinate of bending moment curve at that point. Contour of girders thus presents an unusual and pleasing appearance. Center span is 120 ft. with two 85-ft. approach spans.—D. E. L.

Thick concrete slab acts well under locomotive test load. C. V. ARMOUR. *Eng. News-Rec.*, June 16, 1932, V. 108, No. 24, p. 851-852.—Tests of r. c. railway bridge slab 2½ ft. thick, spanning a main thoroughfare in London, Ontario, and carrying Can. Nat. R. tracks, have provided some results, unique and significant, since these tests are believed to be first in which a modern engine and tender have been used as loading element. Lateral distribution of load throughout slab was surprisingly good, and impact effect was negligible so far as stress increase was concerned, although appreciable vibrations were observed on test dials.—D. E. L.

Eliminating initial stresses in concrete arch bridges. H. E. STEINBERG. *Conc. Constr. Eng.* (England), June, 1932, V. 27, No. 6, p. 326-28.—Construction of bridges with permanent hinges has practical objections. Various methods are used to eliminate secondary stresses in concrete arches. Use of temporary hinges suggested by Considere proved cheap and practical method and first used in Great Britain in 1910, is not suitable for masonry-covered bridges. Constructing arch in independent halves with thin separating steel plates at crown and niches left for hydraulic jacks, makes possible control of stress condition in arch, initial stresses due to shrinkage,

elastic deformation and movement of centering. This method is being employed for first time in England in Chiswick Bridge over River Thames. In Pont de la Caille shrinkage and elastic deformation were eliminated by constructing arch partly of voussoirs of large pre-cast concrete slabs. By varying number and distance of slabs at different points in intrados and extrados manner of deformation of arch was controlled, so as shortening of arch took place curvature altered and distance between springings remained theoretically constant. Further method consists in constructing arch by concreting strips at intervals some distance apart and when these have hardened filling in gaps by depositing additional concrete.—J. M.

Some considerations in the design of hingeless arches. C. V. WOLFF. *Conc. Constr. Eng.* (England), July, 1932, V. 27, No. 7, p. 382-97.—Purpose of paper is to consider degree of error introduced in approximations used in analysis of concrete arches, with reference to 3 factors: variation of moment of inertia of arch between springings and crown, effect upon arch of direct compressions caused by thrust and method of integrating terms involving *ds*. Fundamental equations are stated. Effect of the ratio of moments of inertia at crown and springing on thrusts and moments due to loading and temperature is shown by curves. One fact seen from study of curves is desirability of designing hingeless arches with fairly large ratio of moments of inertia at springings and crown.—J. M.

Construction of Chiswick bridge. W. L. SCOTT. *Conc. Constr. Eng.* (England), June, 1932, V. 27, No. 6, p. 319-26.—Arch vaults were designed self-supporting and capable of supporting superstructure. Temporary staging was designed to carry only weight of arch vaults. Initial and secondary stresses were balanced by hydraulic jacks at crowns of arches. Hydraulic jacks have advantages over hinges in continuous vaults and when bridge is to be faced with stonework. For each of 3 arches 10 jacks of 250 tons capacity were used. Good surface to arch soffit was maintained by placing thin slabs at bottom of jack pockets, casting slabs with arch vaults. Allowances were made for deformation of staging due to weight of concrete arches. Use of hydraulic jacks economized design by eliminating permanent bending moments from arch settlements, and by lifting arches off temporary staging.—J. M.

The design of concrete arches in Allegheny County. G. S. RICHARDSON. *J. Am. Conc. Inst.*, June, 1932, V. 3, No. 10, *Proc.* V. 28, p. 637-652.—Factors favoring concrete arches in considering 6 bridges on Ohio R., Blvd. and George Westinghouse bridge in Allegheny Co., Pa., included lower maintenance cost and saving in re-use of steel centers for several spans. Boulevard spans range from 150 to 400 ft., carry 40-ft. roadway and two 8-ft. sidewalks. Distinctive feature is method of treating approach panels between end and arch abutments. Uniform panel spacing was maintained from backwall to backwall. Arch ribs have a rise of about 1 to 4.5 in proportion to span. Westinghouse bridge consists of 5 arch spans including 460-ft. central span. Arch ribs are 14 ft. wide with crown depth 1/80 of span. Axes of arch ribs conform closely to dead load polygons and are 3-centered curves for shorter spans and 5-centered for longer. Stresses in arch ribs are divided into 2 broad groups —direct stresses resulting from dead and live load, and stresses due to arch shortening. As result of extensive model studies for main Westinghouse span to determine effect of stiffness of superstructure on rib stresses, and best location for expansion joints, bridge was built with joints at ends of spans, model studies indicating a reduction of 40% in live load moments with this design. Live load stress is relatively small proportion of total, ranging from 15 to 25% for shallow ribs. Arch shortening stresses may become more than 50% of total for deep ribs.—D. F. J.

BUILDINGS

New construction of a municipal building in Kassel, Germany. D. BORKOWSKY. *Zement* (Germany), June 23, 1932, V. 21, No. 25, p. 367-72.—R. c. skeleton structure consists of 6-story front section, 2 story back section and 5-story side wing. Upper stories are cantilevered 3 ft. over street. Dead and live loads of various structural parts and members are cited. Foundations of building and special bottom slab construction of vaults are of special interest. Ceilings are of r. c. rib type 10 in. thick.—A. E. B.

Reinforced concrete office building in Frankfurt a. M. H. CRAEMER. *Beton u. Eisen* (Ger.), July 5, 1932, V. 31, No. 13, p. 197-198.—In new headquarters of German Federation of Labor, special attention was paid to insulation against

sound and vibration. Bids were received on both structural steel and reinforced concrete frame. Latter construction showed 18% lower cost.—A. A. B.

Sound insulation in buildings (Ueber den Schallschutz durch Baukonstruktionsteile). H. Reiher. Publ. by R. Oldenbourg Verlag, Munich, Ger., 1932.—A short presentation of theoretical and practical side of insulation against sound. Much of material is based on author's experiments as director of Sound and Heat Research Dept. at Technische Hochschule in Stuttgart.—A. A. B.

Maximum moments in multi-story frames. W. S. Gray. *Conc. Constr. Eng.* (England), June, 1932, V. 27, No. 6, p. 338-44.—Derived equations are given for moments in 2 and 3-story frames. Uniform load is considered and moments derived in terms of uniform loads for different spans, so max. moments are obtained by inspection. Method of slope deflection is used in derivation.—J. M.

Red Cross hospital in Berlin-Wilmersdorf, Germany. Otto Bartning. *Zentralblatt der Bauverw.* (Germany), March 30, 1932, V. 52, No. 14, p. 157-64.—Large hospital structure, is of concrete-encased steel skeleton type and light-weight concrete blocks made from concrete with pumice aggregates.—A. E. B.

Airplane hangar of the seaplane harbor in Ostia, Italy. *Cemento armato* (Italy), 1932, No. 3, 3 p.—General description of r. c. hangar with 2-hinged roof members spanning over 98.4 ft. Hangar was built on pile foundation. Design and construction details are included.—A. E. B.

Factory of 19 stories over railroad yard. *Eng. News-Rec.*, July 7, 1932, V. 109, No. 1, p. 1-4.—Starrett-Lehigh manufacturing and distributing center in New York City, completed in 1931, involves several unusual structural layouts. Railroad tracks on ground floor necessitate widely spaced columns and heavy girders are required in 3rd story to establish uniform column spacing in floors above. Building is concrete slab structure resting on structural steel base 3 stories high above street. Most distinctive feature of building is cantilever-wall construction by which "wall" columns on all floors are set back 8 ft. 9 in. from windows which form continuous bands around building.—D. E. L.

Miscellaneous

The degree of safety of concrete. Neumann. *Cemento armato* (Italy), 1932, No. 5, 2 p.—Fundamental considerations for determination of degree of safety in designing concrete and r. c. structural members and structures are outlined.—A. E. B.

Chart determines shear functions in concrete beams. R. F. Jensen. *Concrete*, July, 1932, p. 21.—A chart for short cuts in design of beams is given together with illustration of its use.—N. H. R.

The moment distribution method. G. S. Coleman. *Conc. Constr. Eng.* (England), April, 1932, V. 27, No. 4, p. 211-218.—Mathematical presentation of method devised by Prof. Cross, Univ. of Ill., gives illustrative examples of continuous beams.—J. M.

Influence of haunches on continuous beams. A. J. Ashdown. *Conc. Constr. Eng.* (England), May, 1932, V. 27, No. 5, p. 284-95.—Calculations for such beams are simplified by series of curves. Following assumptions are made in calculating moment of inertia: moment of inertia is constant between haunches; neutral axis has constant ratio to depth; full steel area at support is to be carried over haunches for at least ½ length of haunch; no steel is considered in compression. Assumptions are justified by assuming tensile stress in concrete is zero. Curves are given for distributed loads and for point load at center of beam, for various ratios of haunch depth to beam depth, of haunch steel area to beam steel area, and for 3 different haunch lengths. Modifications are given in application for T-beams.—J. M.

Simplifying reinforced concrete beam design. Odd Albert. *Eng. News-Rec.*, June 16, 1932, V. 108, No. 24, p. 860-861.—Several design charts presented may be used to advantage in estimating and designing r. c. members. Data presented are for rectangular concrete beams reinforced either for tension or for both tension and compression.—D. E. L.

Stresses in reinforced concrete walls due to non-uniform temperatures. E. Rausch. *Beton u. Eisen* (Ger.), June 20, 1932, V. 31, No. 12, p. 184-191.—In a study of stress development and analysis in chimneys, cooling towers, closed frames and other r. c. structures exposed to unequal temperatures and unable to expand freely, formulas are derived with charts and examples to illustrate their use.—A. A. B.

Load on turbine foundations. A. TROCHE. *Beton u. Eisen* (Ger.), July 5, 1932, V. 31, No. 13, p. 204-207.—A resumé of methods advocated by Geiger, Rausch, Moersch, Kayser and author for determining horizontal and vertical static loads equivalent to dynamic loads acting on turbine foundations. Static horizontal force of 3 times and static vertical force of 5 times machine weight is assumed at center of shaft. This empirical assumption is based on examination of existing foundations. Other methods, all very briefly considered, bring up various factors, vibration, speed, etc. Modern practice uses about half values recommended by Moersch.—A. A. B.

New process for raising dikes, etc., by means of water impervious reinforced concrete structures. MURALT. *De Ingenieur* (Holland), 1932, No. 8, 9 p. —Article illustrates fundamental viewpoints and construction methods for enlarging and raising dikes and dams with r. c. Various cross-sections are discussed and design of expansion joints explained.—A. E. B.

New Czechoslovakian concrete code. F. LAUSER. *Beton u. Eisen* (Ger.), July 5, 1932, V. 31, No. 13, p. 209-210.—Summary of the main points of the new code.—A. A. B.

Theoretical design of concrete mixes. S. STEUERMANN. *Beton u. Eisen* (Ger.) July 20, 1932, V. 31, No. 14, p. 220-227.—In brief summary of some 1930 publications by Construction Research Institute of Tiflis, by mathematical treatment a study of basic principles is made and formulas set up. This is done from 2 viewpoints: first (direct method) considers properties of fine and coarse aggregate in various proportions; second (indirect or mortar method) considers concrete as composed of coarse aggregate and mortar.—A. A. B.

Report of First Polish Reinforced Concrete Convention. Publ. by Assn. of Polish Portland Cement Manufacturers, Warsaw, 1931. Reviewed in *Beton u. Eisen*, July 20, 1932, V. 31, No. 14.—First Reinforced Concrete Convention of Polish Cement Council held in November, 1931, was attended by 400 delegates. Report covers convention and exhibition. Among the 39 papers presented are: X-Ray studies of reinforced concrete members; Form of test specimens (in which the cylinder is advocated); Welding of reinforcement; Effect of clay admixture on shrinkage.—A. A. B.

Development of statics of reinforced concrete with regard to properties of material used. FERNAND CAMPUS. *Preprint, First Congress Int. Assn. for Br. and Str. Eng.*, Paris, France, May, 1932, 18 p.—Design of reinforced concrete structures as a whole agrees but roughly, synthetically, with physical properties of constituent materials. Any difference between theoretical computations and actual strength must be small. Question arises whether increasing knowledge of physical properties of materials demands a fundamental revision of methods of designing reinforced concrete structures, or if only a readaption of experimental data would prove sufficient. Actual increase in strength of concrete is first considered, then influence of elasticity and plasticity of concrete on stability of complete structures, taking into account internal as well as external forces. With some minor reservations the author decides on validity of existing methods in which elasticity is assumed. He also considers effect of strains that are independent of external forces, and local effects of plasticity (self-relieving).—AUTHOR'S SUMMARY.

Co-ordination of basic principles of concrete mixtures. JOS. A. KITTS. *Concrete*, June, 1932, V. 40, No. 6, p. 14-16.—Absolute volume is total volume within surfaces of particles in case of solids, and volume, at usable temperatures, of water and liquid admixtures. It is generally called apparent volume by concrete physicists, and is volume on which standard apparent specific gravity of cement is based. A practical test example given to determine physical characteristics of aggregates, essential to the employment of absolute volume basis, serves to show new basis of density, moisture content and absorption. July, 1932, V. 40, No. 7, 9-11.—Summary of qualities required of concrete, particularly that centrally produced, made under technological process, also elements affecting quality, including types of aggregates and other materials, tests, and research.—C. B.

Benefits from trial mix. STANLEY M. HANDS. *Rock Products*, July 2, 1932. V. 35, No. 13, p. 20-22.—Trial mixes often show better workability and strength result from increase in fine aggregate. Field engineer should be given a range of proportions from which to select that best adapted to his materials and conditions.

In California highway concrete is designed by absolute volume, in effect same as designing by w/c ratio. A table of mixes is given, all with 19.9 cu. ft. aggregates per yd., which with 6 sacks per yd. specified equals a w/c ratio of 0.79-0.81. Percentage of fine in total aggregate varies from 34 to 42%.—E. S.

The mortar voids method of designing concrete mixtures. MARK MORRIS. *J. Am. Conc. Inst.*, Sept., 1932, V. 4, No. 1, *Proc.* V. 29, p. 9-26.—No well-defined method has been generally accepted for selecting combination of materials to compose batch to produce concrete of given strength. Effects of differences between field and laboratory steps upon composition of concrete require certain assumptions during progress of designing and require considerable experience with class of concrete to be made. Relationships between cement, voids, strength of mortar and concrete have been extensively investigated by Iowa State Highway Commission. Relationship established by work of Talbot and Richart was found to exist for all sands available for use in Iowa, and results are in accord for basic laws of composition of mortars and concrete and relationship between their unit strength and their composition. Details of the 2 methods are practically the same.—D. F. J.

WATER WORKS

Roofs and floors of elevated circular tanks. W. S. GRAY. *Conc. Constr. Eng.* (England), April, 1932, V. 27, No. 4, p. 221-227.—In certain cases beamless slab or dome results in final saving in cost for elevated water tank roof and floor. Prejudice against domed floors and roofs is mainly among contractors. Method of design is applied to hemispherical shaped dome but general principles considered apply to any shape such as conical dome. Curves give coefficients plotted against slopes of tangent to dome. For given point coefficient is selected and by substitution in formulae stresses are obtained. Neglecting resistance to bending, equations are derived for stresses in tanks of both hemispherical and conical bottoms.—J. M.

ARCHITECTURAL DESIGN

Telegraph office and a municipal building in Mainz, Germany. G. FREUND. *Zentralblatt der Bauverw.* (Germany), Apr. 6, 1932, V. 52, No. 15, p. 169-73.—Both buildings are of r. c. skeleton type. Special precautions were necessary for protection of concrete foundation of telegraph office due to large amounts of acid in ground water. Details of buildings and architectural designs notable.—A. E. B.

Clinic for nervous diseases in Frankfurt a. M., Germany. MARTIN ELSAESSER. *Zentralblatt der Bauverw.* (Germany), March 2, 1932, V. 52, No. 10, p. 109-15.—Description of layout and architectural design of large r. c. skeleton building group—A. E. B.

Restaurant of the airport Halle-Leipzig, Germany. HANS WITTWER AND L. E. REDSLOB. *Zentralblatt der Bauverw.* (Germany), March 16, 1932, V. 52, No. 12, p. 133-7.—Reinforced concrete skeleton of main building and terrace consists of 6 transverse frames. Basement is provided with slab girder ceiling. Roof structure is flat concrete slab made from concrete with pumice aggregates. Article describes architectural design and character of structure consisting of concrete and glass—cf. *Zement* (Germany), May 12, 1932, V. 21, No. 19, p. 276-8.—A. E. B.

The airport restaurant Halle-Leipzig, Germany. HANS. WITTWER. *Zement* (Germany), May 12, 1932, V. 21, No. 19, p. 276-8.—Modern and very practical architectural design of 2-story building consisting entirely of r. c. and glass is remarkable. Ceiling, roof and walls are supported by row of 6 T-shaped r. c. members. View is unrestricted in all directions. Large terrace with r. c. stairs is built along 1 side of building.—A. E. B.

The new stadium of the City of Florence, Italy. PIER LUIGI NERVI. *Baugilde* (Germany), Apr. 25, 1932, V. 14, No. 8, p. 397-400.—Recently finished stadium with facilities for 32,000 people is r. c. structure. Grandstand with seats for 5000 people is protected by cantilevered r. c. roof without any columns and a span of 72 ft. Remarkable design and aesthetic character of structure are critically discussed and compared with other modern and antique Roman stadiums.—A. E. B.

Six-dollar housing possible through design economy. *Eng. News-Rec.*, July 7, 1932, V. 109, No. 1, p. 9-10.—Designs recently completed for Chicago project by engineers and architects retained by Portland Cement Association demonstrate what can be done in providing low cost group housing. Economies in space layout, structural design and arrangement of equipment resulted in an estimated cost of

$6 per room per month including cost of land. Structural elements, developed after a number of comparative cost studies, include monolithic concrete walls with 3-in. cinder-concrete block lining, painted inside and outside; non-bearing partitions 3-in. thick of 8 x 16-in. cinder-masonry blocks, painted on both sides; solid concrete floor slabs, with 1-in. cement floor finish.—D. E. L.

FIELD CONSTRUCTION

BRIDGES

New construction of 1970-ft., 144-ft. and 174-ft. railway bridges in Copenhagen, Denmark. NIELSEN. *Ingenioren* (Denmark), 1932, No. 13, 8 p.—Description of construction of large building project includes soil investigations, pile foundations, constructive design. Superstructures consist of continuous r. c. frame with hinges supporting 2 tracks.—A. E. B.

Reinforced concrete bridge across the Pesipe River. *Cemento armato* (Italy), 1932, No. 2, 2 p.—Description of construction, design, foundations and cross-sections of r. c. highway bridge spanning river, with openings 98.4 and 65.6 ft. Bridge is 21.3 ft. wide.—A. E. B.

Hanbook for reinforced concrete construction, arch bridges. MELAN AND GESTESCHI. 1932, V. 11, 4th edition. Reviewed in *Zement* (Germany), July 7, 1932, V. 21, No. 27, p. 400.—Third part of treatise deals with hinges of various design, expansion joints and joints at abutments, covering of joints, drainage of bridges, shrinkage phenomena, reconstruction and repair jobs, enlargements, reinforcing of steel bridges with concrete and r. c. Fourth part illustrates construction, design and history of various concrete and r. c. arch bridges with continuous arches. Section 5 deals with r. c. bridges with girder slab cross-section.—A. E. B.

Use structural plywood forms on Illinois River bridge. *Concrete*, Aug., 1932, V. 40, No. 9, p. 8-9.—Plywood form work was used in construction of Illinois R. bridge approach having r. c. piers with large arched openings in diaphragm walls. Architectural treatment was given outer supporting columns of piers. Greater part of plywood used was ¾-in., 5-ply stock, in 3 x 8-ft. sheets, handled similar to wooden form boards. Three-ply stock ⅜ in. thick was used for curved face.—C. B.

The construction of concrete arches in Allegheny County. V. R. COVELL. *J. Am. Conc. Inst.*, June, 1932, V. 3, No. 10, *Proc.* V. 28, p. 653-664.—Construction of 7 r. c arch bridges was supervised by Allegheny Co. Bur. of Bridges in connection with Ohio River Blvd. and relocation of a section of Lincoln Hy., Pittsburgh. Arch spans ranged from 150 to 460 ft. Forms for arch ribs and spandrel columns were transferred from one arch to next one built, and steel centering permitted interchange by slight modification. For winter concreting, high early-strength cement was used in arch ribs, protected by tarpaulins and perforated steam pipes. In building abutments of 460-ft. Westinghouse span, a 400 ft. chute delivered concrete to receiving hopper. Centering for superstructure was placed by cableways and after one rib had attained sufficient strength, rib centering was lowered and slid into position for other rib—D. F. J.

BUILDINGS

Glass and concrete. *Gewapend Beton* (Holland), 1932, No. 8, 6 p.—A brief illustration of examples of combination of concrete and glass for construction of windows, etc.—A. E. B.

Enlargement of main building of food storage house in Leipzig-Plagwitz, Germany. HANS EISLER. *Zeitschr. Oster. Ing. und Archit. Vereins* (Austria), March 25, 1932, V. 84, No. 11-12, p. 55-6.—Article presents brief description of construction of 6-story warehouse of r. c. skeleton type, 66 x 603 ft. and of 5 and 6-story office building, 46 x 288 ft.—A. E. B.

An example of modern dwelling house manufacture. FRITZ MELISSER. *Zeitschr. Oster. Ing. und Archit. Vereins* (Austria), June 17, 1932, V. 84, No. 23-24, p. 122-3.—Construction of large group of dwelling houses erected in suburb of Berlin (Germany) used latest mechanical equipment and well-designed concrete distribution layouts. Houses are built of light-weight concrete made from cement, gravel and blast furnace slag.—A. E. B.

College and home for women in Brunn, Czechoslovakia. BOHUSLAV FUCHS. *Zentralblatt der Bauverw.* (Germany), May 4, 1932, V. 52, No. 19, p. 217-25.—Construction and architectural details of modern school building are presented. R. c. skeleton consists chiefly of 3 frames in longitudinal direction of building. Women's

home, built entirely of concrete, is divided into a number of equal sections easily and quickly erected.—A. E. B.

DAMS

Dam over the Dnieper, Russia, and its power installation. *Genie Civil,* July 16, 1932, No. 2605, p. 53-58.—Complete description of Dnieprostroi project, second in size only to Hoover dam.—M. A. C.

The construction of the Piano Barbellino dam. PEDRETTI. *L'Energia Elettrica* (Italy), 1932, No. 4, 12 p.—Paper describes construction of concrete arch dam, 220 ft. high, layout of construction site, proportioning and mixing of concrete materials, large scale distribution of concrete, precautions against water permeability of dam, foundations and details of design.—A. E. B.

MISCELLANEOUS

Details of field inspection of concrete construction. D. B. RUSH. *Concrete,* Aug. 1932, V. 40, No. 8, p. 22-23.—Duties of field inspector stated in logical order, from checking of cement storage to inspection of curing. Special attention to steel reinforcement is stressed.—C. B.

A Dutch concrete pump. *Gewapend Beton* (Holland), 1932, No. 7, 5 p.—Description of design and application of Kooyman system concrete pump explains composition of most suitable concrete mixture and properties of finished concrete. —A. E. B.

The Torkret process for the construction of high pressure boilers. FRITZ BINSWANGER. *Zeitschr. Oster. Ing. und Archit. Vereins* (Austria), Feb. 26, 1932, V. 84, No. 7-8, p. 33-4.—Protective coatings for high pressure boilers against hot gases consist of layers of refractory concrete sprayed upon reinforcing wire mesh.—A. E. B.

35th main meeting of the German Concrete Society. *Zement* (Germany), Apr. 7, 1932, V. 21, No. 14, p. 207-8.—Brief review of papers presented: Henninger, The construction of the Schluchsee power station; Burckas, The foundation jobs with compressed air and the concrete jobs for the engine house of the Rheine power station Albbruck-Dogern i. B., Germany; Enzweiler, Construction of the hydro power development on the Dnieper, Russia, and Progress in the application of the pressed concrete process for engineering structures; Rudolph, The construction of the Bleiloch dam; and Finsterwalder, the construction of warehouse No. 59 in Hamburg.—A. E. B.

Providing an elevator for Carlsbad Cavern tourists. F. A. KITTREDGE AND W. A. ATTWELL. *Eng. News Record,* June 9, 1932, V. 108, No. 23, p. 821-2.—Passenger elevator installed by National Park Service for eliminating 750-ft. climb to major features of Carlsbad Cavern in New Mexico has shaft 6 ft. 10 in. by 14 ft. 3 in., lined with gunite. Thickness of lining varies from ¼ in. to several inches. A 1:3 mix with 4% calcium chloride was placed under pressures varying from 40 to 125 lb. High-early-strength cement was used for walls and to cover I-beams to permit resumption of drilling an hour after pouring. No segregation of material was noted despite distance through which gunite was pumped. About 20% was lost through rebound.—G. M.

The penstock of the water power development Mezzocorona at the Nice River, Italy. RICCABONA. *L'Energia Elettrica* (Italy), 1932, No. 4, 10 p.—Details of design and construction of 16 ft. wide penstock include data about geologic conditions excavations and linings. Pongaiola Valley is crossed by pressure pipe supported upon r. c. bridge. Observations and measurements taken during construction are presented.—A. E. B.

The Twenthe Canal, Holland. WENTHOLT. *De Ingenieur* (Holland), 1932, No. 22, 20 p.—Description of present state of construction of large Dutch water regulation program includes locks at Zutphen, their location, design and dimensions, design of lock chamber and gates, lock heads with bridge foundations, operation of locks and auxiliary structures, water regulation near Bolksbeek, culvert near Schipbeek and structures near Eefde.—A. E. B.

The construction of the lock and culvert near Zutphen, Holland. EGGINK. *De Ingenieur* (Holland), 1932, No. 23, 4 p.—Details include foundation of lock, pile and steel sheet driving, concrete forms and concreting of floor and walks of lock.—A. E. B.

Concrete lagging lines open caisson for skyscraper foundation. *Concrete*, Aug., 1932, V. 40, No. 8, p. 13-14.—Reinforced concrete lagging used in caisson construction resembles T in cross section. Tops or flanges of T's are placed against earth, interlocked longitudinally by tongues and grooves. Lower portion of T projects inwardly and increases in width as it extends away from flange. Unit becomes integral part of concrete cylinder, retains earth as excavation proceeds, and acts as a form for concrete deposited later.—C. B.

The best method of concrete proportioning and the sand-wetting process. ERICH WEISE. *Zement* (Germany), Apr. 7, 21, 1932, V. 21, No. 14, 16, p. 202-4, 233-7.—After critical consideration of all factors influencing concrete proportioning (the measuring of cement and aggregates, effect of changes in gradation, effect of changes of natural moisture content, effect of wetting of sand and measuring of gauging water), for effective concrete control it is necessary that cement and aggregates be measured by automatic scales. Inundation process is advantage. Amount of water for concrete mixing needs continuous regulation. Relation between w/c-ratio and 28-day concrete strength is pointed out.—A. E. B.

Concrete water control apparatus. C. L. BROCK. *Rock Products*, July 2, 1932, V. 35, No. 13, p. 44-45.—New method and apparatus have been devised for making rapid determinations of moisture in aggregates, necessitated by speed of work in modern ready mixed concrete plant. Apparatus consists of instrument resembling hydrometer, with special scale and hook on lower end on which to hang a metal immersion pan containing sample to be tested. With 500g. of moist sample in pan, apparatus is placed in water. If specific gravity of sample is known moisture may be determined at once from reading of scale. A chart shows straight line relation between specific gravity and figures on moisture scale. Method is accurate within 0.2-0.5%—E. S.

Making, curing and testing of concrete cylinders. D. B. RUSH. *Concrete*, July, 1932, V. 40, No. 7, p. 15-16.—Cylinder moulds should be made of metal, absolutely to size, true and smooth, with attached metal base, to conform as far as consistent with A. S. T. M. specifications and still be suitable for field use. Commercial testing laboratory has 3 sizes made up. Concrete should be placed in 3 to 4-in. layers, each layer puddled with 25 strokes of ⅝-in. rod. Fog room curing permits proper interpretation being placed upon ultimate strength. Properly made cylinders, cured in the laboratory, check mix design, as well as aggregate and cement.—C. B.

ROADS AND PAVEMENTS

How highway departments utilize high-early-strength cement. *Concrete*, Aug., 1932, V. 40, No. 8, p. 5-7.—Twenty-eight out of 43 states reporting have had experience with use of high-early-strength cement. Four of the remaining 15 states report satisfactory results with use of increased quantities of ordinary portland cement for special purposes. Most frequently mentioned purposes in using h. e. s. cement are reduction of curing period, elimination of long detours, closing short gaps in pavements, paving of intersections, and paving roads in regions of heavy traffic.—C. B.

State highway design standards for paving one traffic lane. E. M. FLEMING, *Concrete*, July, 1932, V. 40, No. 7, p. 5-8.—Single-track pavement costs about half as much as 2-lane construction. A 9 or 10-ft. thickened edge design of 9-6-9 in. cross section containing edge bars is recommended by state of Illinois for county use. Most counties use off-center location. When center-line location is used, section is given a crown of ½ in. Maryland, Missouri and Delaware standards are outlined and call for 9-7-9 to 7½-6½-7½ cross section and width of 10 or 9 ft. with roadway widths of 30 to 26 ft.—C. B.

Bids for the construction of concrete roads. P. W. SCHARROO. *Betonstrasse* (Germany), Apr. 1, 1932, V. 7, No. 4, p. 41-4.—Handling of bids for construction of concrete pavements in Germany, Holland, Hungary, Switzerland and other European countries is discussed and advantages and disadvantages of various procedures pointed out.—A. E. B.

The development of concrete road construction. P. W. SCHARROO. 1931, Moormanns Periodieke pers. (Holland), 3rd edition, 123 p., Reviewed in *Betonstrasse* (Germany), Apr. 1, 1932, V. 7, No. 4, p. 53-4.—New edition illustrates rapid develop-

ment of concrete roads in Holland and other countries giving theory and practical examples with appendix: directions for the construction and maintenance of concrete roads.—A. E. B.

The cement macadam road in the Czechoslovakia. KAREL VALINA. *Betonstrasse* (Germany), May-June 1932, V. 7, No. 5-6, p. 63-7; (in German with Czech summary).—Highway 3 miles long was constructed in 3 sections. Details include properties and consumption of road materials, method of building.—A. E. B.

The concrete road in Belgium. R. DUTRON. *Betonstrasse* (Germany), Apr., May-June, 1932, V. 7, No. 4, 5-6, p. 46-52, 57-60.—Twenty-ft. slab of concrete road between Gheel and Het Punt, Belgium, is divided into two 10-ft. sections by longitudinal joint. Slab is 6.7 in. thick in center and thickened over 30 in. to 8.6 in. at edges. Two 30-in. concrete strips for bicycles are built on sides of roadslab. Porphyry aggregates and high early strength portland cement were used. Concrete was poured in 1 course. Highway between Brugge and Ostende, was widened and provided with new top concrete over stretch of 7 miles. New road is 27 ft. wide, slab 7 in. thick at center and 9 in. at edges. Transverse joints at intervals of 40 ft. were filled with not bituminous material. Road machinery is described.—A. E. B.

Cement macadam roads in Leimen near Heidelberg, Germany. W. HAUG. *Betonstrasse* (Germany), May-June 1932, V. 7, No. 5-6, p. 62-3.—Old road was lowered about 10 in. to bring subgrade to required elevation. Embankments were faced with concrete blocks. A 5-in. layer of broken stone was compressed with 8-ton roller upon loamy subsoil. On this was placed 2-in. layer of mortar (1:3) and 5-in. layer of broken stone (1-1½ in.).—A. E. B.

Concrete road construction in Austria. MARTIN SCHEFFEL. *Betonstrasse* (Germany), July 15, 1932, V. 7, No. 7, p. 75-8.—Austria possesses great natural resources for excellent hard stone aggregates and quartz gravels suitable for concrete pavements, but financial conditions of country do not permit a large road building program. Construction of concrete pavement in Ebensee (Austria) is described.—A. E. B.

Laying small granite block pavement in concrete. J. SCHULZE. *Betonstrasse* (Germany), July 15, 1932, V. 7, No. 7, p. 81-2.—In new type of pavement small granite blocks, about 3 in. high, imbedded in concrete form durable wear-resisting surface with good grip for vehicles.—A. E. B.

Brick-concrete road, an economic pavement for medium and heavy traffic of any nature. HIRT. *Betonstrasse* (Germany), May-June, July, 1932, V. 7, No. 5-7, p. 67-71, 83-5.—This type of pavement needs solid and well-compressed subgrade, an intermediate concrete layer to distribute uniformely occurring loads, a layer of bricks laid flat upon concrete with well filled joints with fat cement mortar, cement mortar coating and well-placed granite or concrete curbstones. Example shows actual practice of designing and placing and cost calculation reveals economic advantages.—A. E. B.

Guildford and Godalming by-pass road. W. P. ROBINSON. *Betonstrasse* (Germany), May-June, 1932, V. 7, No. 5-6, p. 61.—Concrete road is 9.5 miles long, 30 ft. wide between curbs. Thickness of roadslab is 8 in. with edges thickened to 10 in., and 10 in. thickened to 12 in. where concrete is laid on fill. One-course concrete is reinforced with 2 layers of mild steel bars, mats being assembled at site of construction. Mix is 1:2:4 of crushed shingle and washed sand. Slab is provided with 1 longitudinal joint along center line and transverse joints at 30 ft. Latter have steel dowel bars. Concrete is cured first with cocoanut matting, then with damp soil for 21 days. Concrete curbs, 10½ x 4½-in. are laid upon upper surface of slab with suitable expansion joints.—A. E. B.

SHOP MANUFACTURE

Advocates trial mixtures for concrete pipe and block. GEO. L. REED. *Concrete*, Aug., 1932, V. 40, No. 8, p. 19.—Method used successfully in South for pipe and products has been to design suitable wet plastic concrete by trial, selecting aggregate combination producing most economical mix, generally a 5-gal. mix for pipe and a 6½ or 7-gal. mix for products. With this combination of aggregates, water is reduced as necessary for particular machine involved and a series run with varying cement contents. Any specification can be met.—C. B.